DETERIORATION OF EXCAVATED ROCKSLOPES: MECHANISMS, MORPHOLOGY AND ASSESSMENT

by

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The candidate confirms that the work submitted is her own and that appropriate credit has been given where reference has been made to the work of others

Mum

1938-1996

This is for you

Rose-coloured glasses and all that

"The stone the builders rejected has become the capstone"

Psalm 118:22

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ABSTRACT

Results are reported of laboratory simulated weathering processes on a range of sedimentary rocks and the investigation of the deterioration of rock masses as observed on engineered and quarried rockslopes.

The simulated weathering processes include freeze-thaw, wetting and drying, salt weathering and slaking. It is found that the rock properties of pore volume, saturation coefficient and microporosity exert greatest influence on susceptibility to breakdown. For stronger rocks it is found that durability correlates well with high strength and elasticity.

A range of rock flaws visible in hand specimen are described and their influence on rock deterioration assessed. Linear flaws such as laminations and stylolites are more likely to be associated with breakdown, and the role of structural weaknesses is most evident in stronger rocks. Rock breakdown mode due to experimental weathering is found to closely resemble material weathering of source slopes in the field. A range of rock breakdown mechanisms are inferred from changes in pore microstructure and rock strength. There are indications of a progression from deterioration which is invisible and involves modification of the existing pore structure, to macro deterioration resulting from generation of new void space and microcracks.

After field investigation of more than two hundred rockslopes deterioration is found to be widespread, and there is little evidence of a systematic approach to its assessment or mitigation. Fracture spacing, rock strength and lithology are found to be the most influential factors in rockslope deterioration and these are used to define a characteristic range of rock mass types. A range of morphological forms attributed to deterioration are defined and described. An engineering classification of deterioration modes is presented, based on constituent material size, velocity of movement and frequency of occurrence. Deterioration modes correlate well with rock mass type.

A new rock mass classification, called Rockslope Deterioration Assessment (RDA) is proposed, dealing specifically with shallow, weathering and erosion-related rockslope processes. RDA is divided into three stages; a ratings assessment of deterioration risk, a qualitative review of the likely deterioration hazard, and guidance on appropriate mitigation. The findings of the experimental work are incorporated into stage one of RDA where appropriate. Notable in this respect is the emphasis in RDA on evaluation of fracture spacing on the basis of all fractures present, whether open or incipient, and whether natural, or induced by blasting, weathering or stress release. RDA is applied to the slopes investigated in the fieldwork and shows that certain types of rock mass are associated with higher risk of failure. There is also an element of predictability in the occurrence of deterioration modes. Correlation between stage one of RDA and Rock Mass Rating is examined and it is shown that, although there are some similarities, a fundamental difference relates to the basis upon which fracture spacing is assessed.

CONTENTS

CHAPTER FOUR CONTROLS AND INFLUENCES ON ROCK DETERIORATION SUSCEPTIBILITY

PART TWO: ROCKSLOPE DETERIORATION – THE MASS SCALE

CHAPTER SIX FACTORS AFFECTING ROCKSLOPE DETERIORATION

CHAPTER SEVEN RESULTS OF THE FIELD INVESTIGATION OF ROCKSLOPE DETERIORATION

<u>CHAPTER EIGHT</u> ROCKSLOPE DETERIORATION ASSESSMENT (RDA)

ROCKSLOPE DETERIORATION ASSESSMENT: STAGE ONE

ROCKSLOPE DETERIORATION ASSESSMENT: STAGE TWO

ROCKSLOPE DETERIORATION ASSESSMENT: STAGE THREE

LIST OF FIGURES

LIST OF TABLES

LIST OF PLATES

LIST OF ABBREVIATIONS

Common units (SI) are given where appropriate.

Abbreviations related to mechanical rock properties

C_o	Unconfined compressive strength (MPa)
D	Specimen diameter (m)
E_{dyn}	Dynamic Young's modulus of elasticity (GPa)
E_{st}	Static Young's modulus of elasticity (GPa)
IS_{50}	Point load strength (MPa)
K_I	Crack tip stress intensity (for mode I failure)
K_{Ic}	Critical stress intensity (for mode I failure)
K_0	Stress corrosion limit
L	Length of direct wave path (m)
T_{mr}	Modulus of rupture (MPa)
Vf	In situ (measured in the field) P-wave velocity (m s^{-1})
Vl	Intact (measured in the laboratory) P-wave velocity (m s^{-1})
Vp	P-wave velocity (compressional waves) (m s^{-1})
Vp_{air}	P-wave velocity of air (m s^{-1})
Vp_{init}	Initial P-wave velocity (m s^{-1})
Vp_n	P-wave velocity measured after *n* cycles of experimental weathering (m s^{-1})
Vp_0	Calculated P-wave velocity (Fourmaintreaux 1976) (m s^{-1})
Vs	S-wave velocity (shear waves) (m s^{-1})
Y	A geometry factor for stress intensity calculations
b	Breadth (diameter) of specimen (mm)
c	0.5 crack length for penny-shaped cracks
d	Mean depth of specimen cross section (mm)
t	Transit time of the generated pulse (μs)
w	Width of fractures (m)
γ	Wavelength of the generated pulse (m)
μ	Elastic shear modulus (GPa)
ν	Poisson's ratio
σ	Normal stress
σ_r	A remote applied stress
τ	Shear strength

Non-quantitative abbreviations

CalS	Calcareous sandstone
F	Factor of safety
FT	Freeze-thaw
HdCh	High density chalk
LamZ	Laminated siltstone
LdCh	Low density chalk
MagL	Magnesian limestone
MetS	Metasediment
MicS	Micaceous sandstone
MS	Salt weathering (magnesium sulphate)
OolL	Oolitic limestone
RDA	Rockslope Deterioration Assessment
RDA_U	Unadjusted RDA rating
RDA_A	Adjusted RDA rating
RQD	Rock Quality Designation
SD	Slake durability
SEM	Scanning electron microscope
SpaL	Sparry limestone
WD	Wetting and drying
WeaS	Weathered sandstone

Abbreviations relating to measurements at the mass scale

A	Area of the block overlying the discontinuity
U	Uplift force due to pore water pressure
V	Cleft water pressure in a vertical crack
W	Weight of the block overlying a discontinuity
α	Angle of dip of the discontinuity plane
c'	Apparent cohesion
ϕ	Friction angle (deg)

Abbreviations related to void-dependent rock properties and deterioration indicators

A	Pre-test oven-dried mass (g) (chapter 3)
B	Post-test oven-dried mass (g)
CD	Crack density (after Peacock 1994a, 1994b)
F	Number of fractures
F_D	Surface area to volume ratio of fractures in a cylindrical rock specimen (mm^2/mm^3)
ID	Deterioration Index (%)
IFp	Index of Fracture Porosity (%)
IQ	Quality Index (Fourmaintreaux 1976)
LRP	Largest remaining piece (%)
M_{disp}	Displaced mass (g)
M_{dn}	Dry mass after *n* cycles of experimental weathering (g)
M_{d0}	Initial dry mass (g)
M_s	Saturated mass (g)
M_{sub}	Submerged mass (g)
P	Applied pressure of intruding mercury (N/m^2)
P_L	Number of point intersection per unit length of grid line
R	Radius of curvature of a meniscus (m)
S	Saturation coefficient
$\overline{S}$	Mean fracture surface area (mm^2)
S_V	Surface area to volume ratio of planes in a material (mm^2/mm^3)
W_{ab}	Water absorption capacity (%)
g	Acceleration due to gravity ($m\ s^{-2}$)
h	Height of capillary rise
n_e	Effective porosity (%)
n_m	Porosity as determined from mercury intrusion porosimetry (%)
n_t	Total porosity (%)
r	Radius of a 2-dimensional circular fracture (m)
v	Volume (cm^3)
γ	Fluid surface tension (g/cm^3)
μn_m	Microporosity (%)
θ	Contact angle (deg)
ρ	Rock density (kg/m^3)
ρ_w	Water density (kg/m^3)

CHAPTER ONE
INTRODUCTION

1.1 General Introduction

Examination of disused limestone quarries in Derbyshire prompted the author to question the applicability of existing weathering and rock mass classification schemes and slope stability analytical techniques for the assessment of the small but frequent falls of rock which were evident. While evaluation of potential *deep-seated failures* is standard practice for excavated rockslopes such as these, scant attention is given to *shallow surface processes* at the design stage and later. This is because rockslope 'deterioration' is often not perceived as a significant risk, it is difficult to quantify and its mechanisms are poorly understood. To close this gap in engineering practice there is a need to define and characterise rockslope deterioration, to gain a more complete understanding of the mechanisms involved and to provide a means by which it can be adequately and systematically evaluated.

Deterioration is used here as an umbrella term to describe the combined effects of physical and chemical weathering and some erosive agents. Deterioration occurs because when a new slope is excavated in a rock mass two fundamental changes occur. First, the act of excavation releases confining pressure on the rock mass leading to expansive recovery (Gerber and Scheidegger 1969; Feld 1976; Nichols 1980). This is achieved by an increase in total void space and largely effected by fracture propagation and dilation (Matheson 1995). Second, excavation exposes a newly-created rock mass surface to ambient environmental conditions (Price 1995), especially temperature and moisture fluctuations.

As a result of these changes, that part of the rock mass which has been exposed by excavation is no longer in equilibrium with its surrounding internal and external environment (Gagen and Gunn 1988). The natural response to this is for equilibrium to be re-established through progressive breakdown and erosion of the rockslope. Excavated rockslopes deteriorate in an *accelerated* fashion (Gunn and Gagen 1987) because of their non-equilibrium state and thus significant change can occur in engineering time. When a rock mass is excavated artificially, by whatever means, release of confining pressure and exposure to the environment occur instantaneously in a geological sense. The excavation techniques used can also induce significant damage to the rock mass (Dubin et al 1986; Ross and Reeves 1995). Thus the rate of excavation for artificial slopes differs from the much longer term denudation typical of natural slopes and there are different mechanisms involved.

Deterioration occurs by the *weathering* of rock material and its subsequent *removal* from the rock mass. The rate, frequency and magnitude of material removal varies considerably. Slope deterioration can be manifest in many ways, including as rockfall, ravelling, granular disintegration, debris slide and the fall of isolated rock fragments (Nicholson et al 2000). Although freefall is involved in most such mechanisms, some element of sliding, toppling and flow can also occur. Deterioration can cause weakening of the rock mass and results in changes to its morphology, including changes of geometry and surface cover, and deposition of material both on the slope and at the foot.

1.2 The Consequences and Engineering Significance of Rockslope Deterioration

Rockslope deterioration poses an engineering problem because (i) it has the capacity to weaken the rock mass and material and to modify the slope morphology; (ii) the spatial and temporal distribution of these changes is difficult to predict; (iii) deterioration is progressive and time dependent, and therefore has the capacity to affect the primary rockslope throughout its design life, and also the efficacy of stabilisation measures and remedial works.

There are several potential consequences arising from rockslope deterioration and some examples are illustrated schematically in Figure 1.1.

1.2.1 Slope maintenance and remedial works

Rockslope deterioration has the potential to generate significant debris, both on the slope and at the foot (Wright 1981; Spang 1987; Hungr and Evans 1988; Gagen 1988; Williams 1990). The debris requires regular clearance in order to maintain toe drains and to maximise the capacity of rocktraps. A build-up of debris at the foot of a slope also has the potential to influence adversely the trajectory of falling material (eg Ritchie 1963; Spang 1987) and can therefore increase the safety hazard (Robotham et al 1995). Deterioration can weaken the rock mass creating areas of instability which might necessitate stabilisation or protective works, or simple maintenance measures such as scaling (Fookes and Sweeney 1976; Fookes and Weltman 1989). These remedial structures can be vulnerable to damage or disruption by ongoing deterioration, calling for regular inspection, repair and replacement as necessary.

In all of these cases it is clear that if maintenance and remedial requirements arising from rockslope deterioration are not addressed at the design stage then there will be continuing, unplanned resource implications. Alternately, resource wastage occurs due to over-design, or by excessive and unnecessary inspection and monitoring. At present, even if the implications of rockslope deterioration on maintenance are addressed at the outset there is no systematic approach available to enable assessment of the likelihood of deterioration, its potential mode (and thus its frequency and magnitude), or its temporal and spatial distribution. Consequently, it will be difficult to target maintenance and remedial works in the most efficient manner.

1.2.2 Safety hazards

Where deterioration is manifest as a freefall of loose material it constitutes a potential safety hazard. Falls of rock in British quarries were responsible for 25 fatalities or serious injuries in British quarries over a 20 year period (Walton 1993b; DETR 2000). The safety hazard is usually perceived as being located primarily at the foot of the slope, but subsidence at the crest and collapse of overhangs might also create a potential danger at the top. The safety hazard can be to people (eg pedestrians, quarry workers, educational groups) (Walton 1993b; Robotham et al

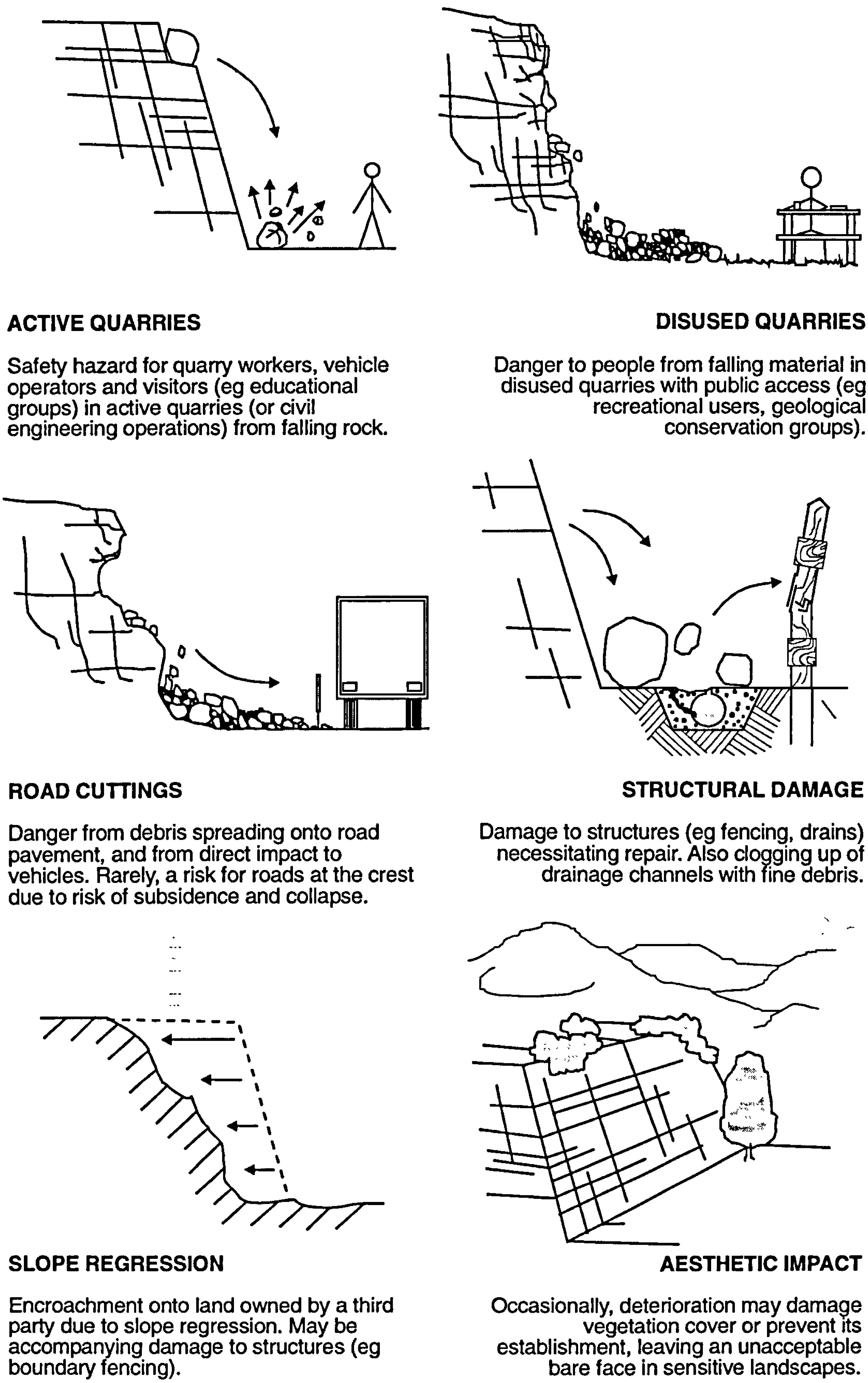


Figure 1.1 Diagrammatic representation of some consequences of rockslope deterioration

1995), to vehicles (eg road and rail transport, quarry machinery) (Martin 1988) and to structures (eg toe drains, fencing, pavements). In many cases it is not difficult to remove or considerably reduce these risks by adopting appropriate design strategies, or with the use of protective and remedial works (Fookes and Sweeney 1976; Fookes and Weltman 1989). With greater understanding of the nature of deterioration and its spatial and temporal distribution the need for such works, their appropriate planning and selection, and their ongoing management could be considerably improved and better targeted.

1.2.3 Morphological change

Deterioration can result in the considerable loss of material from rockslopes leading to significant modification of the slope morphology in engineering time (Gagen 1988). Re-distribution of material within a slope and deposition at the foot can also have a similar effect. There are several implications arising from this:

(a) Boundary modification

A receding crest-line or build-up of debris at the foot of a slope can cause encroachment onto land owned by a third party. This can be particularly problematic where this causes a quarry to stray beyond a mineral extraction planning permission boundary, or where the encroachment disrupts a neighbouring land use (eg Department of Environment, Transport and the Regions 2000).

(b) Aesthetic impact

Modification of slope morphology and the re-distribution of detached material can have a significant influence on the overall appearance of the slope landform. In a particularly sensitive landscape this can be critical (eg Nicholson 1995). An illustration of this would be where progressive deterioration limited the establishment of vegetation on the face of a disused quarry, thus preventing successful assimilation of the slope into the surrounding landscape (Glen 1985). Re-design of quarry slopes using restoration blasting and landform replication techniques has been used to reduce this problem in some sensitive landscapes (Gagen and Gunn 1987a,1987b,1987c; Gagen 1988; Gagen and Gunn 1988; Wakefield et al 1992; Gagen et al 1993). Conversely, modification of slope surface morphology and material weathering are likely to enhance the appearance of rockslopes (Haywood 1974), particularly where this means camouflaging production or pre-split drillhole sequences.

(c) Conservation issues

Build-up of talus might obscure features of interest at geological conservation sites (Nature Conservancy Council 1990; Moseley 1990). Complete *loss* of a feature of interest is unlikely to occur in hard rock quarries or road cuttings since these are more usually conservation sites of the 'exposure' rather than the 'integrity' type (Department of the Environment 1996), nevertheless, it remains a possibility.

1.2.4 Other consequences of rockslope deterioration

In cases where excavated rockslopes must be re-excavated after a period of disuse or inactivity, such as re-opening of a quarry or widening of an existing road, deterioration which has occurred in the interim might influence the subsequent excavatability of the rock mass. In many cases this consequence is likely to be beneficial since deterioration will weaken the rock mass rendering it more easily excavatable. Nevertheless, this consequence warrants consideration since it will have implications for the method of excavation used, the time required and the financial costs involved.

1.3 Current Understanding of Rockslope Deterioration

Deterioration occurs at the rock mass and material scales and involves both chemical and physical processes. Chemical weathering is usually manifest as mineral alteration, decomposition and dissolution and does not commonly occur at sufficient scale in the temperate climate of the UK to produce wholesale weakening of a rock mass (Saunders and Fookes 1970). However, local modification of rock material can be significant and exposure of palaeo-weathered profiles can also occur. The outcome of physical weathering is the rupture of rock material by the generation and development of fractures (Fookes et al 1988). This is the case at the rock mass scale where stress relief generates rebound fractures, at the microscopic scale where frost or salt weathering can induce intragranular and grain boundary cracks and at all scales in between. Chemical processes such as stress corrosion also contribute to fracture initiation and propagation (Whalley et al 1982). The characteristics of fractures and their inter-relationships are a major control on the size, shape and spatial distribution of detached material on a rockslope which is available for re-distribution or removal. In situ decomposition by chemical weathering also weakens the rock material, preparing it for transport. The *mode* of rockslope deterioration is determined by a combination of material rupture and weakening and is a critical influence on the consequences of deterioration.

Given the relatively limited and localised importance of chemical weathering in the UK, there will be greater emphasis in this research on physical weathering processes, notably the role of rock fracture in deterioration. Much has been published on the mechanics of fracturing (eg Ingraffea 1987; Atkinson and Meredith 1987; Murakami 1987; Aliabadi 1999), the characteristics of rock fractures (eg Chernyshev and Dearman 1991; Ameen 1995), and the application of that knowledge to the understanding of the role of fractures in deep-seated slope instability (eg Hencher 1987; Bell 1992a; Richards 1992). However, studies specifically looking at the role of fractures and small scale flaws in rock weathering in the laboratory (eg Smith and McGreevy 1983) and at the field scale (eg Douglas 1981; Douglas et al 1994) are relatively rare. There is also a general lack of understanding of how fractures relate to different *modes* of deterioration at all scales.

These gaps in our current understanding of the fundamental mechanisms involved in rockslope deterioration and their relationship with the nature of material removal from slopes need to be addressed. In addition, despite the fact that numerous classifications and assessment schemes for slope instability, landslides, slope processes and rock mass properties have been produced (eg Varnes 1958; Carson and Kirkby 1972; Hoek 1973; Hutchinson 1988; Dikau et al 1996), there are none which specifically consider progressive, near-surface weathering and erosion of excavated rockslopes. This also needs to be addressed.

1.4 Current Approaches to the Evaluation of Rockslope Deterioration

Many of the current approaches to the assessment and classification of rock masses (see Geological Society Engineering Group Working Party 1977; Bieniawski 1989; Bell 1992b) and their stability (Hack and Price 1993) contain elements which could be used in the evaluation of rockslope deterioration. However, since none of these methods were specifically designed for that purpose they have a number of inadequacies and limitations in this respect. For example, the Rock Mass Rating system devised by Bieniawski (1976, 1979, 1993), and subsequently modified by Romana (1988, 1993) for use in slope stability assessment, was designed for modes of failure which depend upon the presence of distinct discontinuity planes. This is also true for kinematic analysis using stereographic projection techniques (eg Hoek and Bray 1981; Walton 1988) and for limit equilibrium calculations (eg Nash 1987; Hencher 1987). In theory, the movement of even a small fragment of rock from a slope will usually involve some element of sliding or toppling and can therefore be analysed using these concepts. However, analysis of the stability of individual rock fragments constituting a large rockfall is not practicable. While larger scale forms of deterioration involve some movement along discontinuity planes, breakdown is more usually independent of them.

A number of weathering classifications have also been published (Moye 1955; Ruxton and Berry 1957; Chandler 1972; Martin and Hencher 1986; Geological Society Engineering Group Working Party 1995), but these were designed to provide a description of the static condition of the rock mass or material as a result of past weathering and do not attempt to address time dependent processes or susceptibility to weathering. They also do not consider the transport phase which, as mentioned previously, is critical in determining the consequences of deterioration and thus its mitigation.

Several slope and rockfall hazard assessment schemes have been published (Sinclair 1992; Bunce et al 1997; Franklin and Senior 1997a, 1997b; McMillan and Matheson 1997, 1998) as well a number of rockfall trajectory studies (Ritchie 1963; Mak and Blomfield 1986; Spang 1987; Spang and Rautenstrauch 1998; Robotham et al 1995). Again, there are elements of each which would be useful in the evaluation of rockslope deterioration, but many were designed for purposes which restrict their applicability in this context. Some schemes, for example, emphasise slope failure by rockfall but do not deal with the much wider range of mechanisms involved in deterioration. Others address natural, rather than excavated slopes, or are applicable to specific conditions of a very limited geographic area. Some schemes emphasise the mechanisms of fall but not its likelihood, consequences or mitigation, and others focus on the risk of rockfall or slope failure but do not distinguish between different failure modes.

In practice, deterioration of rockslopes is often dealt with on an ad hoc basis, with those responsible identifying remedial requirements as the need arises. Identification of need is commonly by infrequent slope inspection and walkover survey. A wide range of in-depth process-oriented investigations have been undertaken by geomorphologists (Luckman 1976; Douglas 1981; Douglas et al 1991, 1994; Matsuoka and Sakai 1999), but while making a valuable contribution to the understanding of fundamental processes, they are generally not designed for direct application to engineering practice. Their direct applicability is further hampered by the wide variety of approaches and techniques used, the lack of a standard terminology and the fact that very few geomorphic studies have been conducted in the context of man-made slopes.

1.5 Research Aims and Objectives

To address some of the issues raised, the broad aim of this research is to investigate the mechanisms and morphology of the deterioration of excavated rockslopes and to design a method for its characterisation and assessment. This aim translates into three core objectives:

Objective one: to investigate breakdown mechanisms and the role of rock fractures and other rock properties on weathering at the material scale and the mode and severity of deterioration which results. This is achieved by subjecting a variety of rock types to simulated weathering processes, also enabling some evaluation of the effects of varying environmental conditions.

Objective two: to determine from field investigation of excavated rockslopes the nature and morphology of deterioration, and to gain a better appreciation of its consequences (some assessment of the intrinsic and external factors influencing and controlling rockslope deterioration will also be attempted).

Objective three: to utilise the results of objectives one and two to develop a rock mass classification which addresses the problem of excavated rockslope deterioration. The aim is to develop a scheme which enables assessment of the susceptibility of a rock mass to deterioration, the modes of deterioration which are likely to occur and their consequences, and which also provides guidance on mitigation of deterioration.

In order to limit the effects of variation in environmental conditions, the field investigation is largely confined to the UK. This means that applicability beyond the UK is restricted to regions with similar climatic conditions. A comprehensive range of rocks is included in the research though in the experimental work there is greater emphasis on sedimentary rocks. Any slope materials which would normally be regarded as soil are excluded and only steep slopes, taken here to exceed 45°, are included. Although most of the rockslopes investigated were either road cuttings or quarry faces (active and disused), the research results are applicable to other types of rockslope such as railway cuttings and temporary slopes associated with civil engineering works.

1.6 Structure of the Thesis

A diagrammatic representation of the main components of the thesis and their inter-relationships is given in Figure 1.2. The thesis is divided into two parts. In Part One deterioration of rock at the *material scale* is considered and the results of the experimental laboratory work are presented and discussed. In Part Two, the deterioration of *rockslopes* is discussed and the results of the field investigation are presented together with the proposed Rockslope Deterioration Assessment method.

In Chapter Two, the fundamental principles of rock deterioration are reviewed with reference to the properties of rock, the weathering environment and rock fracture mechanisms. The term *deterioration* and implications for this work are briefly discussed. In Chapter Three, the experimental weathering programme is presented, together with descriptions of the four processes used: freeze-thaw, wetting and drying, salt weathering and slake durability. The sampling regime, measurement of deterioration and determination of rock properties are also described. A lithological description is given of the ten laboratory samples.

In Chapters Four and Five, the results of the laboratory investigation are presented and discussed. In Chapter Four the emphasis is on quantifying the severity of rock deterioration and on identifying relationships between rock properties and deterioration. The relative merits of different deterioration indicators are reviewed and a brief assessment also made of the role of environmental conditions in rock deterioration. In Chapter Five the emphasis is on the mode of breakdown and the mechanisms involved. Rock flaws are described and their role in breakdown investigated. Micro-scale mechanisms of breakdown are explored through analysis of the modifications to rock properties which have been induced by experimental weathering.

In Chapter Six, a wide range of intrinsic and external controls and influences on rockslope deterioration are reviewed. In Chapter Seven, data collection for the field investigation is described. The results of the field investigation are presented, beginning with an overview of the occurrence, consequences and mitigation of deterioration in the UK. Classifications of deterioration morphology, deterioration modes and rock mass types are presented and described in outline.

In Chapter Eight the key findings of the experimental and field investigations are brought together and a new rock mass classification called Rockslope Deterioration Assessment (RDA) is presented. A ratings approach to the assessment of rockslope susceptibility to deterioration is presented in stage one of RDA. Ratings adjustments are described which allow consideration of a wide range of external influences. In stage two of RDA the nature of deterioration is addressed using more detailed accounts of the classifications introduced in Chapter Seven. In stage three of RDA mitigation of deterioration is discussed, with guidance on general approaches and detailed treatment measures appropriate to the assessments made in stages one and two.

In Chapter Nine, RDA is applied to the rockslopes investigated in the field and the results are presented and discussed. A number of worked examples are provided and the practical application of RDA evaluated. A comparison is made between RDA and the Rock Mass Rating system and the implications discussed. In Chapter Ten, the key conclusions of the thesis are presented, together with some suggestions for further research.

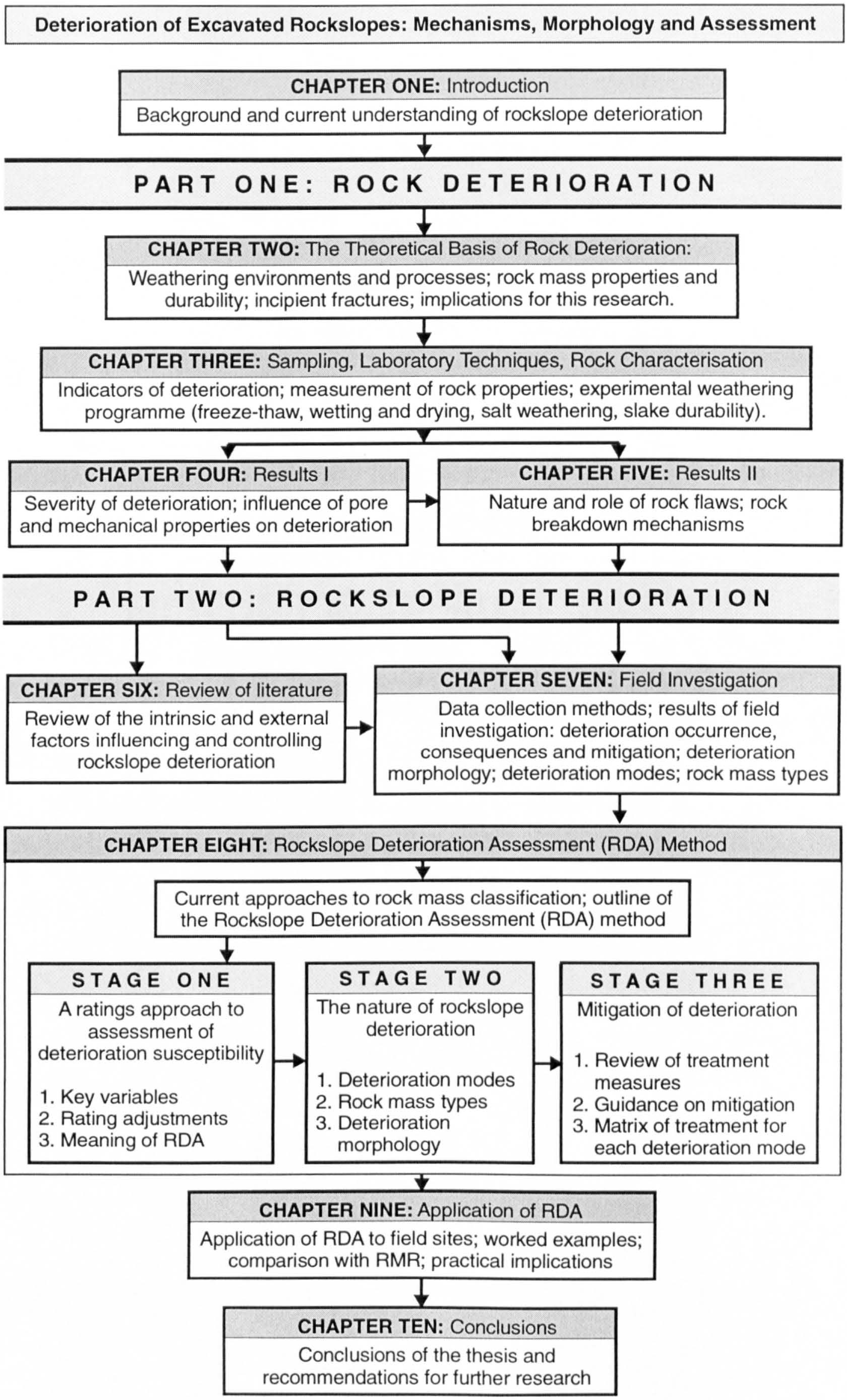


Figure 1.2 Outline structure and principle components of the thesis

PART ONE

ROCK DETERIORATION THE MATERIAL SCALE

CHAPTER TWO
THE THEORETICAL BASIS OF ROCK DETERIORATION

2.1 Introduction

This chapter begins Part One of the thesis, where the focus is on rock deterioration at the material scale. The aim in this chapter is to review and comment on published literature concerning rock weathering, with respect to (i) rock material properties, (ii) the weathering environment, and (iii) the interactions between these two (following McGreevy 1982). Coupled relationships between influences and controls on deterioration, including mass properties and external factors, are indicated in the interaction matrix given in Figure 2.1 (following the method of Hudson 1991, 1992). Some of these interactions are discussed further in Chapter Six. Although the emphasis in this chapter is on deterioration of rock material, reference will be made to deterioration at the rock mass scale for completeness and clarity.

2.1.1 Definitions and terminology

In this thesis deterioration is defined as: *the small-scale shallow, progressive, physical and chemical alteration of material in a parent rock mass, its detachment and subsequent removal or re-distribution by transport agents.* This definition of deterioration and its relationship with other terms used in the literature such as slope stability, rockfall, weathering and erosion will be explored further below.

2.1.1.1 Slope instability

It is widely acknowledged that discontinuities largely control the stability of rock masses and considerably reduce rock mass strength below that of the intact rock material (Hoek 1973). The properties of these discontinuities, particularly their spacing, and orientation with respect to the slope plane, will dictate the likelihood and mode of failure (Hoek 1973; Matheson 1985). In the context of slope stability analysis the discontinuities of interest are those which dissect intact rock at the rock mass scale, including bedding planes, joints and faults (Hoek 1973). Limit equilibrium analysis can be used to model the forces acting on these major joint sets and stereographic modelling of slope stability is based on the geometric relationships between them (Phillips 1971; Matheson 1983a; Hencher 1987; Nash 1987; Selby et al 1988). Rock mass classification systems have also emphasised the importance of large scale discontinuity spacing and orientation (Bieniawski 1979; Romana 1993). The fundamental condition for failure occurs when the forces driving failure exceed the resisting forces, and when the relationship between slope and discontinuity geometry allows for movement of material above the potential failure plane. The types of failures considered in terms of slope instability tend to be of considerable volume and occur along a distinct, identifiable failure plane. These failures also tend to be deep-seated although clearly shallow instability occurs (Symons 1970; Perry 1989; Reid 1998). Four distinctive failure modes, planar (or translational), wedge, toppling and circular, are generally recognised (Hoek 1973; Hoek and Bray 1981) (Figure 2.2). Several authors also indicate a further mode, that of rockfall (Walton 1988; Richards 1992), while others indicate the potential

Rock Mass Structure (eg massive, fissile, blocky, composite) **1.1**	Rock mass structure determines spatial distribution of variations in fracture characteristics 1.2	Rock mass structure determines spatial distribution of lithological variations 1.3	Rock mass structure influences surface area exposed to external conditions 1.4	Variations in permeability and porosity coincide with variations of rock mass structure 1.5	Discontinuous rock masses have highly variable stress field 1.6	Rock mass quality affects choice of excavation method; excavation complex in composite rock masses 1.7	Slope morphology influenced by heterogeneities of rock mass structure 1.8	Different stabilisation methods suit different types of rock mass structure 1.9	Rock mass type closely associated with time dependent mechanisms (eg flexural toppling in fissile rock masses) 1.10	Deterioration mode closely associated with rock mass type (eg grain ravelling in weak massive slopes) 1.11
Characteristics and distribution of fractures have a major influence on rock mass structure 2.1	**Fracture Characteristics** (aperture, spacing, infilling, continuity) **2.2**	Rock material is ruptured by fractures at all scales (eg microcracks, tectonic joints) 2.3	Open fractures allow ingress of moisture, temperature fluctuations, and organic matter 2.4	Flow pathways and flow velocity determined by fracture aperture, infilling and distribution 2.5	Direction and magnitude of *in situ* stress field controlled by fracture properties 2.6	Fracture spacing, aperture and distribution constrains choice of excavation method 2.7	Fracture geometry controls large scale slope morphology and designed slope height, angle and benching 2.8	Individual bolting for large blocks; small blocks might require surface protection and improvement (shotcrete) 2.9	Stresses are dissipated along fractures, thereby reducing the potential for time dependent relaxation 2.10	Some deterioration modes are defined by block size and shape (eg stone and blockfall, slabfall, toppling) 2.11
Rock strength and lithology and their distribution have a major influence on rock mass structure. 3.1	Strong rocks more resistant to fracture propagation. 3.2	**Rock Material Properties** (compressive strength, lithology, texture etc) **3.3**	Rock lithology, texture, pore properties and solubility determine susceptibility to material weathering 3.4	Texture and nature of intergranular matrix/cement define primary porosity and permeability 3.5	Fresh, unweathered rock is more highly stressed and therefore prone to residual stress release and rebound 3.6	Strong rocks need to be excavated by blasting; strength, density and sonic velocity determine rippability 3.7	Stable slopes in strong rocks can be steep and high 3.8	Weak rocks are more likely to require stabilisation than strong rocks, which are more self-supporting 3.9	Deformable intact rock material can creep 3.10	Weak rocks prone to granular breakdown and alteration; strong rocks prone to blocky disintegration 3.11
Weathering weakens rock mass and targets boundary zones between lithological or structural variations 4.1	Fractures propagate, enlarge and extend due to weathering (eg freeze thaw, salt weathering and root wedging) 4.2	Weathering leads to material weakening (eg by mineral alteration, dissolution and disintegration) 4.3	**Weathering and Environment** (temperature, exposure, moisture and altitude) **4.4**	Atmosphere provides primary inputs into hydrological system (eg precipitation, evapo-transpiration) 4.5	Erosion reduces gravitational stresses, leading to rebound; weathering releases residual stresses 4.6	Rocks with a high clay mineral content can deteriorate dramatically upon exposure 4.7	More conservative slope dimensions needed for severe environmental conditions 4.8	Exposure, moisture and temperature regime control success of vegetation establishment 4.9	Fatigue failure of rocks occurs due to long term variations in weather conditions (eg freeze-thaw, wet and dry) 4.10	Weathering processes are a fundamental contributor to rockslope deterioration 4.11
High surface runoff can be erosive, causing gullying; groundwater flow in soluble rocks can form karst 5.1	Fracture aperture can be changed by removal of infill by groundwater flow, or by water erosion 5.2	Intact rock material weakened under saturated conditions 5.3	Presence of water is a vital component in most weathering processes 5.4	**Hydrologic Conditions** (groundwater, surface runoff, drainage) **5.5**	Water in pores increases effective stress 5.6	Some explosives are not waterproof (water control might be required); mechanical excavation difficult in wet conditions 5.7	Slope morphology changed by erosive surface runoff (eg gullying, karstification) 5.8	Groundwater flow and surface runoff determine slope drainage requirements 5.9	Water absorption causes cyclic shrinkage and swelling, especially in mudrocks 5.10	Some deterioration modes (eg dissolution, wash and debris flow) depend upon the presence of water 5.11
In situ stress state influences general tightness or looseness of rock mass 6.1	Stress release creates new fractures due to rebound and spalling 6.2	Release of residual stress at grain boundary scale causes fracture of intact rock 6.3	High normal stresses can cause fracture closure, reducing the effectiveness of weathering 6.4	High in situ stress field reduces primary and secondary permeability 6.5	**Static and Dynamic Stresses** (gravitational, ground vibration, air blast) **6.6**	Controlled blasting or mechanical excavation could be necessary in high *in situ* stress field 6.7	Pre-split blasting creates more planar slope morphology when slope perpendicular to σ_3 and parallel to σ_1 6.8	Lateral slope retention and support (eg buttressing) required where horizontal stresses high 6.9	Low *in situ* stresses might allow creep in some rocks (eg shales) 6.10	Displacements of rock can be triggered by dynamic (eg quarry blasting) or static stresses (eg rockburst) 6.11
Hand and mechanical excavation cause less disruption to rock mass structure than blasting 7.1	Blasting induces new fractures; changes block size and causes dilation of existing fractures 7.2	Intact rock quality deteriorates with proximity to the excavated surface 7.3	Excavation exposes the rock mass to the atmospheric environment 7.4	The slope can act as a 'sink' due to reduced fluid pressure, with flow from the exposed rockslope 7.5	Excavation reduces confining pressure and can lead to mobilisation and re-distribution of *in situ* stresses 7.6	**Rock Mass Excavation** (excavation method, loss of support) **7.7**	Pre-splitting creates more planar slope morphology than bulk blasting 7.8	Pre-split, hand or mechanically excavated slopes require less post-excavation treatment 7.9	Progressive deterioration of the rock mass is accelerated upon excavation 7.10	Potential failure surfaces daylight after excavation 7.11
Slope orientation determines exposure of rock mass features such as bedding, shear zones and intrusions 8.1	Slope height affects evaluation of fracture persistence; orientation affects fracture daylighting 8.2	Intact rock properties can be scale dependent 8.3	Frost cycles, prevailing wind direction, precipitation and solar radiation determined by slope aspect 8.4	Rough slope surface reduces surface runoff velocity and inhibits water erosion 8.5	Greater release of confining stress occurs in deeper excavations 8.6	Shallow angle slopes are difficult to create by blasting 8.7	**Slope Geometry and Morphology** (slope angle, height, orientation and form) **8.8**	Higher, steeper slopes require more effective protection against falling debris 8.9	Greater slope equilibration time required for larger, deeper excavations 8.10	Large scale deterioration modes (eg rockslide, rockfall and rock avalanche) possible on large slopes 8.11
Reinforcement changes a discontinuous rock mass into a continuum with greater rock mass strength 9.1	Rockbolts inhibit fracture dilation; fractures can be sealed and drained 9.2	Rock material protected by surface cover, but vegetation can lead to local material breakdown 9.3	Shotcrete protects rock surface from atmospheric weathering environment 9.4	Crest, slope and toe drainage reduce and control surface runoff and groundwater flow 9.5	Reinforcement by bolting locally increases confining pressure and rock mass strength 9.6	Remedial excavation works may form part of maintenance operations locally 9.7	Large scale support can mask true slope form; stabilisation allows for higher, steeper slopes to be created 9.8	**Engineering Factors** (slope stabilisation, protective works and maintenance) **9.9**	Rebound inhibited by slope support and reinforcement 9.10	Appropriate treatment stabilises the rock mass, reducing progressive deterioration and deep-seated failure 9.11
Creep and slope relaxation cause displacements and deformation of the rock mass 10.1	Displacements due to creep and rebound are taken up along existing and new fractures 10.2	In a continuing cycle fresh rock is exposed to atmospheric conditions as weathered material is removed 10.3	Woody vegetation, a potential influence on rock breakdown, requires time to colonise and become established 10.4	Ongoing deterioration changes flowpaths by altering fracture distribution and aperture 10.5	Time dependent processes result in gradual release of stresses until equilibrium reached 10.6	*Rate* of excavation for man-made slopes is considerably higher than that for natural slopes 10.7	Expansive recovery and creep processes lead to an increase in rock mass volume 10.8	Stabilisation measures have finite design life, so renewal, repair and maintenance are necessary 10.9	**Time Dependent Behaviour** (creep, relaxation and cyclic fatigue) **10.10**	With time, deterioration could ultimately lead to deep-seated failure (monitoring might enable prediction) 10.11
Deterioration leads to general weakening and loosening of rock mass structure 11.1	Displacements cause increase of fracture aperture and density 11.2	Deterioration results in the alteration, detachment, removal and re-distribution of rock material 11.3	A weakened rock mass structure becomes more vulnerable to weathering processes 11.4	Rock mass deformation and displacement results in change of hydrologic conditions at all scales 11.5	*In situ* stresses and stress concentrations reduce with ongoing deterioration 11.6	Long term deterioration might necessitate slope cleaning, scaling, re-grading and re-excavation 11.7	Slope angle and height are reduced by deterioration 11.8	Deterioration can affect treatment measures (eg granular disintegration around boltheads) 11.9	Deterioration occurs in engineering time 11.10	**Rockslope Deterioration Response** (shallow deterioration, deep-seated failure) **11.11**

Figure 2.1 Interaction matrix of lithological, environmental and engineering controls and influences on rockslope deterioration, following the method of Hudson (1991, 1992)

How to read the matrix: The primary variables are included along the main diagonal of the matrix, and the rows and columns are numbered 1 to 11.
The influence of the subject in Box 7.7 on the subject in Box 9.9 is shown in Box 7.9 and vice versa for Box 9.7.
The matrix is not comprehensive but most of the key interactions are shown.

role of time dependent weathering processes in slope instability (Hoek 1973; Anderson and Richards 1987; Williams 1990). Other very large forms of slope failure, more usually described in terms of landsliding (eg Dikau et al 1997) are also recognised, including various flow, creep and avalanche mechanisms.

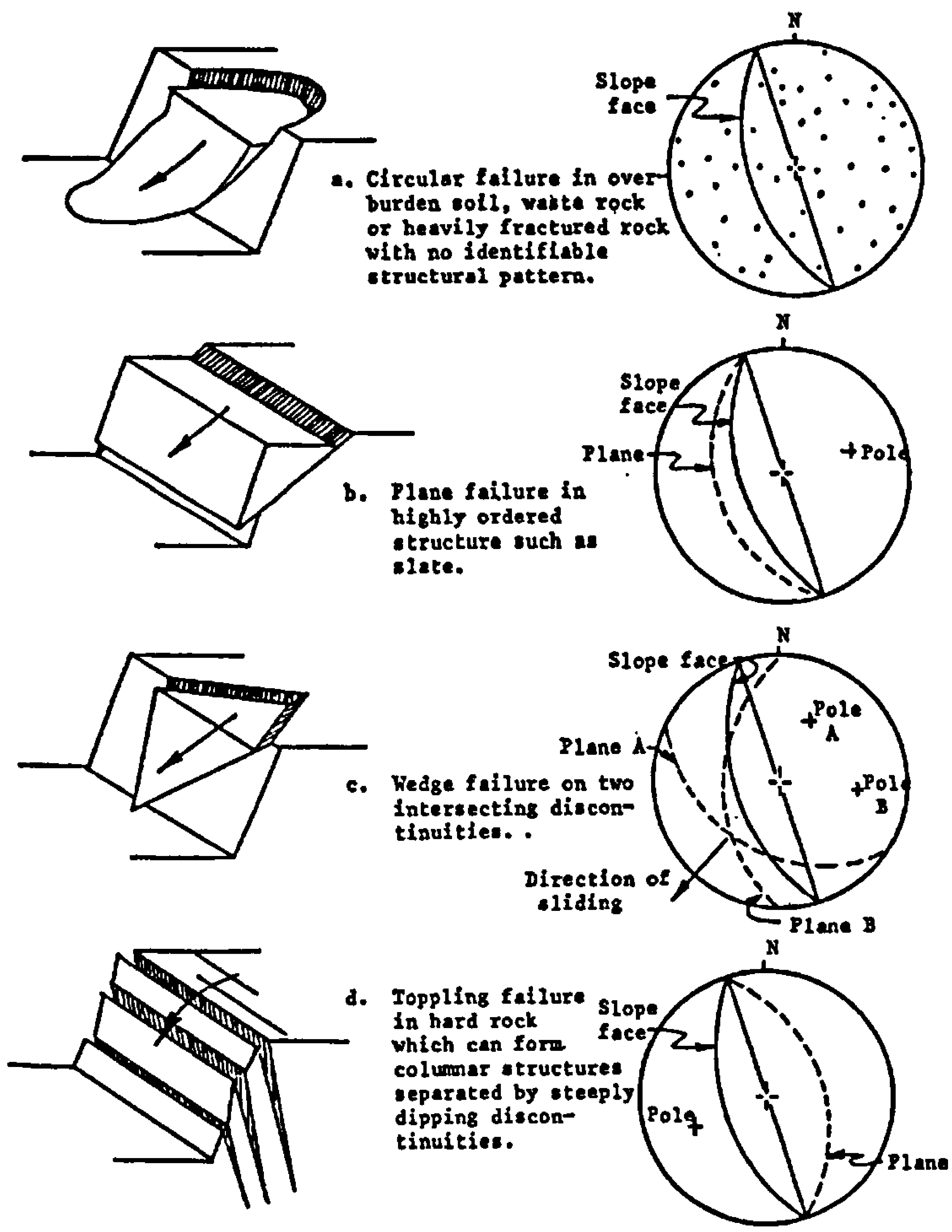


Figure 2.2 Major slope failure modes together with lower hemisphere equal area stereoplots of discontinuity data (*after Hoek 1973*).

2.1.1.2 Rockfall

Rockfalls are usually regarded as being shallower and smaller in volume than other modes of failure (Robotham et al 1995) though they can cause significant problems for cut slopes (Spang 1987; Spang and Rautenstrauch 1988; Richards 1992). The mechanisms involved in rockfall are poorly defined in the literature but it is generally agreed that weathering plays an important role in preparing a rock mass for this type of failure (Whalley 1984; Selby et al 1988; Williams 1990; Selby 1993; Robotham et al 1995). It is notable that the rare attempts to distinguish between different types of rockfall have usually been on the basis of magnitude rather than form (Rapp 1960; Whalley 1984; Selby 1993), either of the constituent material or of the total volume of the rockfall. The magnitude can vary from the fall of isolated blocks to substantial falls of the rock mass (Whalley 1984) and as Selby (1993) notes, it is unlikely that the same fundamental mechanisms will operate at all scales. For example, large magnitude, deep-seated rockfalls described as *rock mass falls* (Selby 1993) are internally fractured masses of rock which fail along a single discontinuity plane. Conversely, particle falls (Selby 1993) are weathering-related, with individual blocks being defined by fractures. Geomorphic studies (eg Rapp 1960; Luckman 1976; Gagen 1988) have shown that rockfall often occurs with considerable frequency, and that despite its relatively low magnitude it is this high frequency which poses greatest problems when in close proximity to developed areas and structures (Richards 1992).

2.1.1.3 Deterioration: weathering and erosion

The defining features of deterioration are that it is (i) weathering-related; (ii) not controlled by failure along a discontinuity plane (though clearly individual blocks are defined by fractures at all scales); (iii) shallow; and (iv) relatively low magnitude. Thus deterioration would not generally include the major types of slope instability referred to above. It also excludes mechanisms where the rock mass fails along a distinct discontinuity plane. It is generally acknowledged that rockfall involves some element of freefall (Dikau et al 1997), though individual block movements usually involve sliding and toppling. Some of the fundamental mechanisms of movement, therefore, are replicated at all scales though the trigger factors for failure in each case might be different. Rock deterioration involves mechanisms which are always shallow, more usually occurring in the outer skin of a rock mass up to 2-3m deep, and occasionally penetrating to 10m in a temperate environment. This partly reflects the depth of penetration of atmospheric influences. The total volume of material involved in deterioration rarely exceeds $20m^3$ and is more typically at least an order of magnitude less, down to the scale of a single mineral grain. Deterioration is progressive in engineering time, whatever the initial condition of the rock mass upon exposure and the fundamental mechanisms of deterioration are the combined effects of physical and chemical weathering processes and erosion.

Weathering is defined by Selby (1993, p123) as:

> "the process of alteration and breakdown of soil and rock materials at the Earth's surface by physical, chemical and biotic processes"

Further definitions are given by Price (1995). Dearman (1974) usefully indicates the three fundamental effects of weathering: (i) discoloration and solution of rock material, primarily by chemical processes, which will not necessarily produce weakening of the rock material; (ii) disintegration of rock material due to grain boundary and mineral grain fracturing, and decomposition of mineral grains, both leading to weakening (Figure 2.3); (iii) weakening of the rock mass by physical and chemical weathering processes, leading to solution, fracturing, granular disintegration and decomposition (Figure 2.4). The all important factor included in Dearman's (1974) definition of weathering is that it is essentially an in situ process, with negligible or no transport occurring. However, in reality, if there was never any removal or re-distribution of weathered material from a deteriorating rockslope the only hazard resulting would be the reduction of rock mass strength and the consequent reduced capacity to withstand a load. The removal of detached material by various transport mechanisms, or erosion (Ollier 1984), such as wind, water, ice and gravity, therefore, forms the second essential component of the term deterioration.

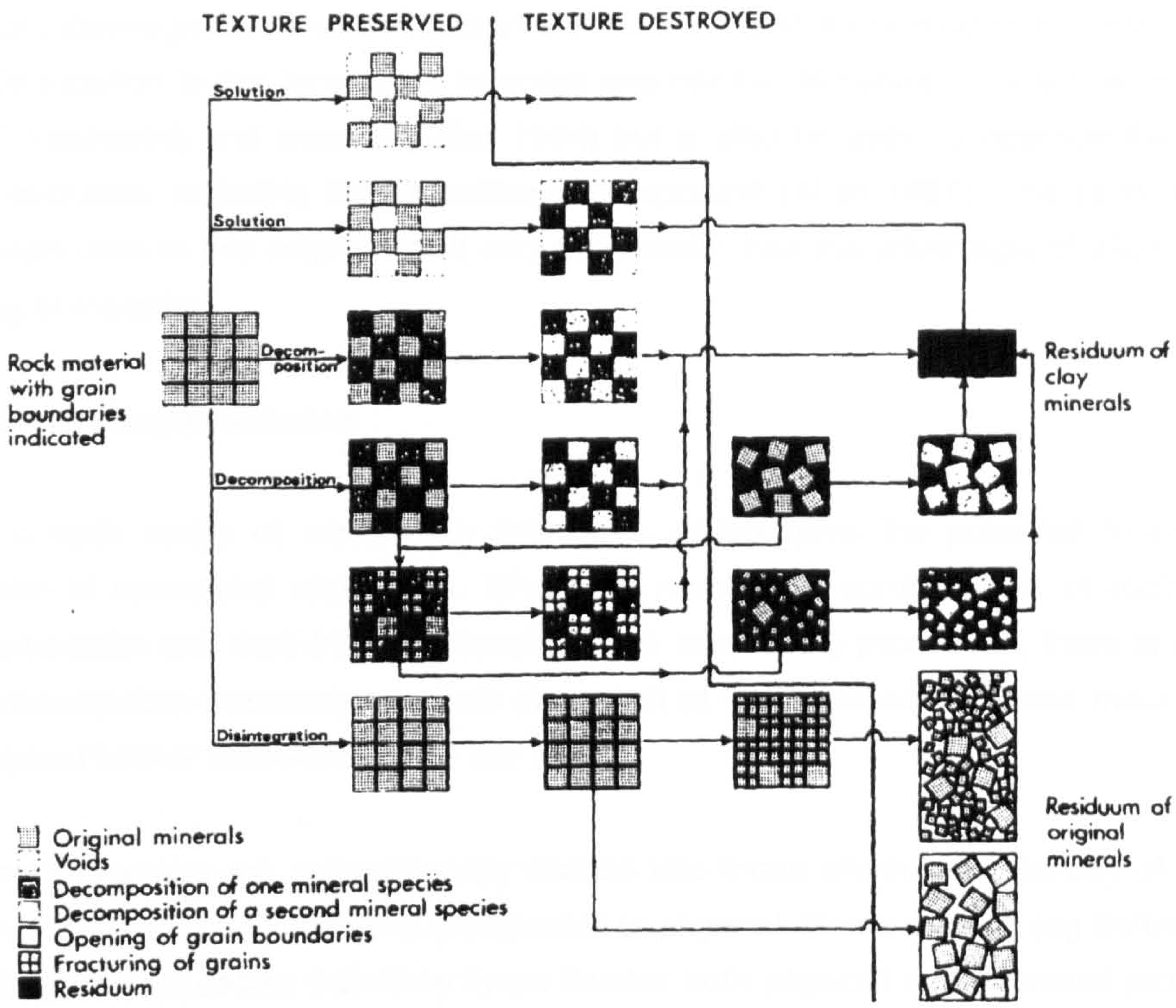


Figure 2.3 A model of the various stages of rock <u>material</u> weathering (*after Dearman 1974*).

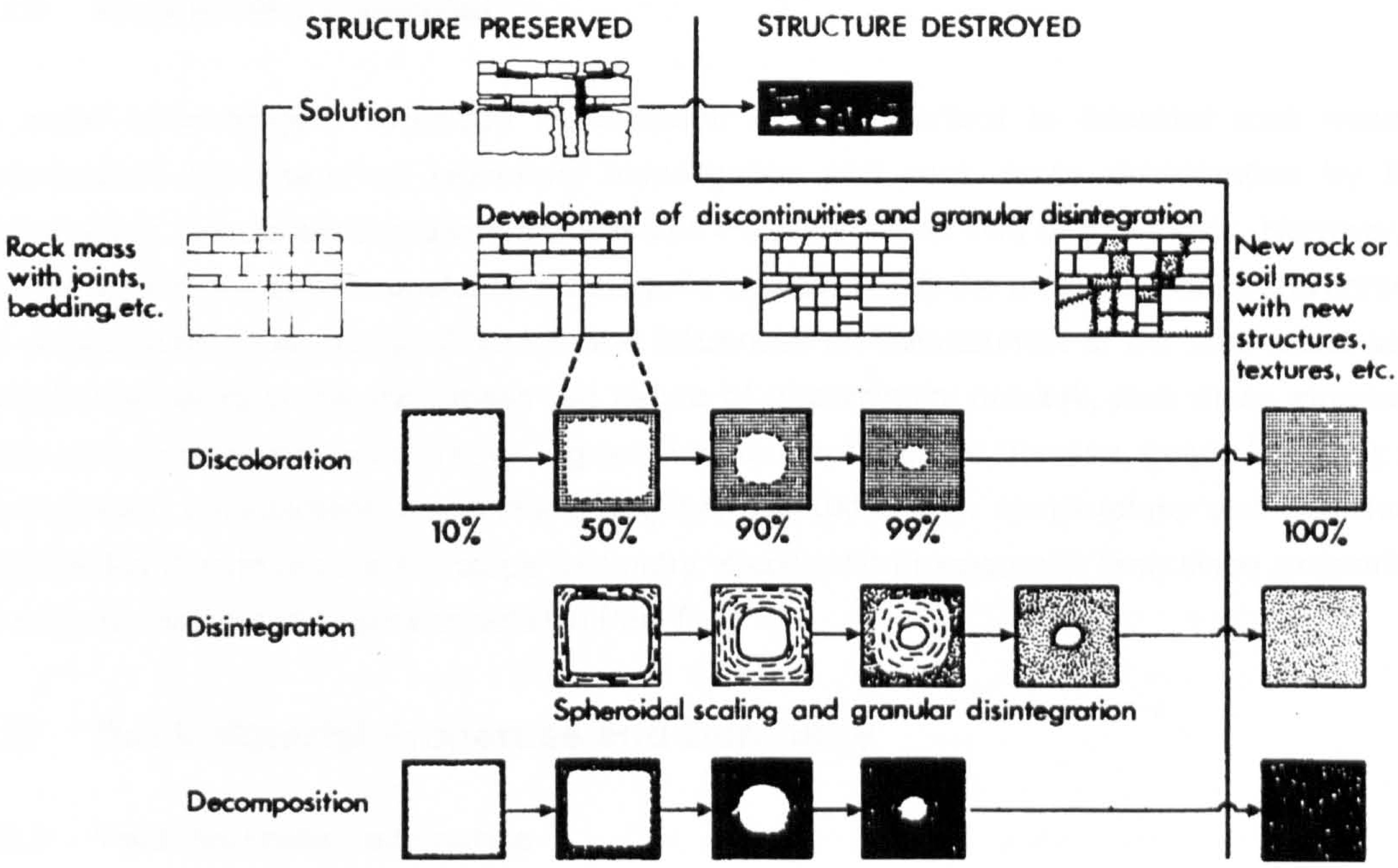


Figure 2.4 A model of the various stages of rock <u>mass</u> weathering (*after Dearman 1974*).

Thus, deterioration is the product of weathering and erosion in which weathering processes lead to alteration, disintegration, and/or detachment of rock material enabling its removal by transport agents. Denudation is the term used in some geomorphic literature to describe the combined effects of weathering and erosion (Ollier 1984) but is also be used to describe the process of landform evolution, including the depositional component (Allen 1997). The term deterioration has not been used in this latter context and additionally, has the advantage of alluding to in situ weakening of material.

2.1.2 Weathering processes

There is a wide range of weathering processes which have the potential to contribute to deterioration of excavated rockslopes. Given the number of combinations of rock mass and material properties and their inter-relationships with weathering processes, there is a very wide range of deterioration mechanisms which can result at the mass scale. These mechanisms are not considered further until Part Two of the thesis.

Weathering processes are conventionally divided into those which are primarily of a chemical nature and those which are primarily mechanical or physical. Some authors (eg Selby 1993) also distinguish biotic processes, but since these involve both physical and chemical processes this is probably unnecessary. This two-fold division of weathering is useful for describing and identifying individual processes, but it is clear that physical and chemical processes can operate simultaneously, or that more than one type of physical or chemical process can act simultaneously (Whalley et al 1982; Jerwood et al 1987, 1990a, 1990b; Atkinson and Meredith 1987).

2.1.3 Rock mass and material

In order to investigate rockslope deterioration it is convenient to consider rock material deterioration by means of laboratory investigation and rock mass deterioration by field investigation. As stated, this division delineates Parts One and Two of this thesis. However, to set the work in context it is useful at this stage to indicate briefly the main influences and controls on deterioration at all scales. Controls and influences on deterioration at the rock mass scale include the nature of the rock mass (eg nature of discontinuity network, rock mass structure); static and dynamic stress conditions (eg residual and gravitational stresses, quarry blasting); the atmospheric environment (eg climatic regime, fluctuations in temperature and moisture); engineering design factors (eg slope geometry, stabilisation measures); time since excavation. These are considered in greater detail in Part Two.

2.2 Rock Material Properties and Durability

2.2.1 Void dependent properties

Porosity describes the volume of void space in rock as a percentage of the volume of the rock. Porosity can vary from near zero in dense, crystalline rocks to more than 60% in loosely consolidated deposits (Domenico and Schwartz 1998). A more typical range for rocks is from 2 to 35%. The amount of intergranular pore space is affected by several other rock properties: *Grain sorting*: In a poorly sorted material (in the geological sense) where a range of grain sizes are represented, voids between coarse grains are more likely to be occupied by fine grains. Thus lower porosity is associated with poorly sorted rock and higher porosity with well sorted rock. *Packing*: A close packing arrangement will leave little void space between grains. *Cementation*: Porosity will be reduced where voids are filled with cementing or detrital (ie transported) materials, or where recrystallisation occurs during diagenesis (Houseknecht 1987). *Grain size and shape*: For rounded grains, higher porosity will be associated with finer-grained rock, though this relationship is often less clear in carbonates (Choquette and Pray 1970). Greater interlocking, and thus lower porosity, is associated with angular grains. In igneous rocks, many of the voids present are likely to be intragranular microcracks.

Porosity of an unconsolidated sediment can be reduced by various processes occurring during diagenesis, such as consolidation during burial leading to grain re-arrangement and deformation (Houseknecht 1987). It is notable that in chalk, where cement contacts between grains can be formed very early in the burial process, porosity (20-40%) is usually very high (Clayton 1983; Bell et al 1990). This is because early cementation makes the material relatively resistant to consolidation (Clayton 1983). Another process affecting porosity is the precipitation of crystals from migrating pore fluids, which can increase as well decrease porosity. Chemical decomposition and alteration of minerals can lead to a reduction in porosity by deposition of alteration products, or can increase porosity by the enlargement of pores and cavities during dissolution. Porosity can also be increased by mechanical weathering processes leading to microcrack propagation and pore coalescence.

It is important to make the distinction between effective and total porosity. Total porosity relates to the total amount of void space within a rock. However, for practical purposes, not all of these pores are connected to each other or to the rock surface and are therefore not able to participate in the storage and supply of water, nor to affect movement of fluids in and out of the rock. Lack of connectivity occurs if pores are completely isolated, if pores or pore throats are too small to absorb fluid, or if they are 'trapped pores'. These are larger pores which are surrounded by very fine pores which are too small to allow the passage of fluid. The total amount of connected void space within a rock, which therefore potentially contributes to the net movement of fluid within the material, is known as effective porosity.

The proportion of accessible and connected pore spaces which actually becomes filled with water upon being immersed for a period of time defines the saturation coefficient. This is a function of several other properties including permeability and capillarity. Permeability describes the ease of fluid flow within a porous medium and is a function of pore connectivity, tortuosity of pore connections, and the overall distribution of pore sizes.

Capillarity is the tendency for fluids to rise in a tube, analogous to pores or microcracks in rock, by an amount which increases with decreasing radius of the column:

$$h = \frac{2\gamma}{gR} \qquad [2.1]$$

Where h = height of capillary rise; γ = fluid surface tension (approximately 0.074g/cm); g = acceleration due to gravity; and R = radius of curvature of the meniscus (comparable to the radius of the tube). Capillarity is also influenced by the wettability of the mineral matter. It occurs because of the surface tension of the fluid which arises due to molecular attraction at the surface. Capillarity enables the migration of fluids and explains how salt attack and frost shattering can occur in unsaturated rock. Capillarity also influences the degree of saturation and explains why higher saturation coefficients are recorded in rocks with finer pores (eg Honeyborne and Harris 1958; McGreevy 1982). Microporosity, saturation coefficient and capillarity are closely related and thus measurement of one provides a reasonable surrogate for the others.

One of the key determinants of the pore size distribution is the percentage of micropores contained in a rock, that is, the proportion of pores less than 1μm (after McGreevy 1982, 1996). It is these pores which have the greatest influence on capillarity and thus absorption of water into the rock. The very finest pores, less than 0.1μm are too fine to absorb water (Winkler 1994). The concept of trapped pores, also known as 'ink-bottle' pores (Fitzner 1988) is further indication of the importance of pore size distribution and particularly the relationship between pores of different sizes in controlling access to the pore structure by fluids. Analysis of modifications to pore size distribution resulting from weathering processes could provide useful insight into the breakdown mechanisms at work.

2.2.2 Lithological and mechanical properties

Rock bulk density is the ratio of dry mass to volume and is a function of the specific gravity of the constituent minerals and total porosity. The constituent minerals and their properties also determine rock hardness which can be related to compressive strength. Schmidt hammer rebound, for instance, measures rock hardness and together with bulk density, can be used to estimate compressive strength (Figure 2.5 after Deere and Miller 1966). Unconfined compressive strength (C_o) is the ability of the rock to withstand an applied load. When the compressive strength of the rock is reached, failure occurs, usually by splitting or shearing. Unconfined compressive strength has generally been used as the index of rock strength. It is reasonably representative of near surface conditions where rocks are not confined by high overburden loads. Compressive strength (C_o) increases in finer grained rock and decreases with increasing moisture content. In anisotropic rocks, measurements made with the applied load perpendicular to structural planes of weakness will be much higher than those made parallel to those weakness planes.

Tensile strength, or a surrogate such as modulus of rupture (T_{mr}), is often used in the stone industry. This is because the most likely mode of failure is in tension due to wind loading and thermal cycles on building panels and cladding, for instance (eg Logan et al 1993b). Both tensile strength and point load strength (IS_{50}) involve elements of compression and tension (unless the former is measured using direct pull tests), and failure is usually by splitting.

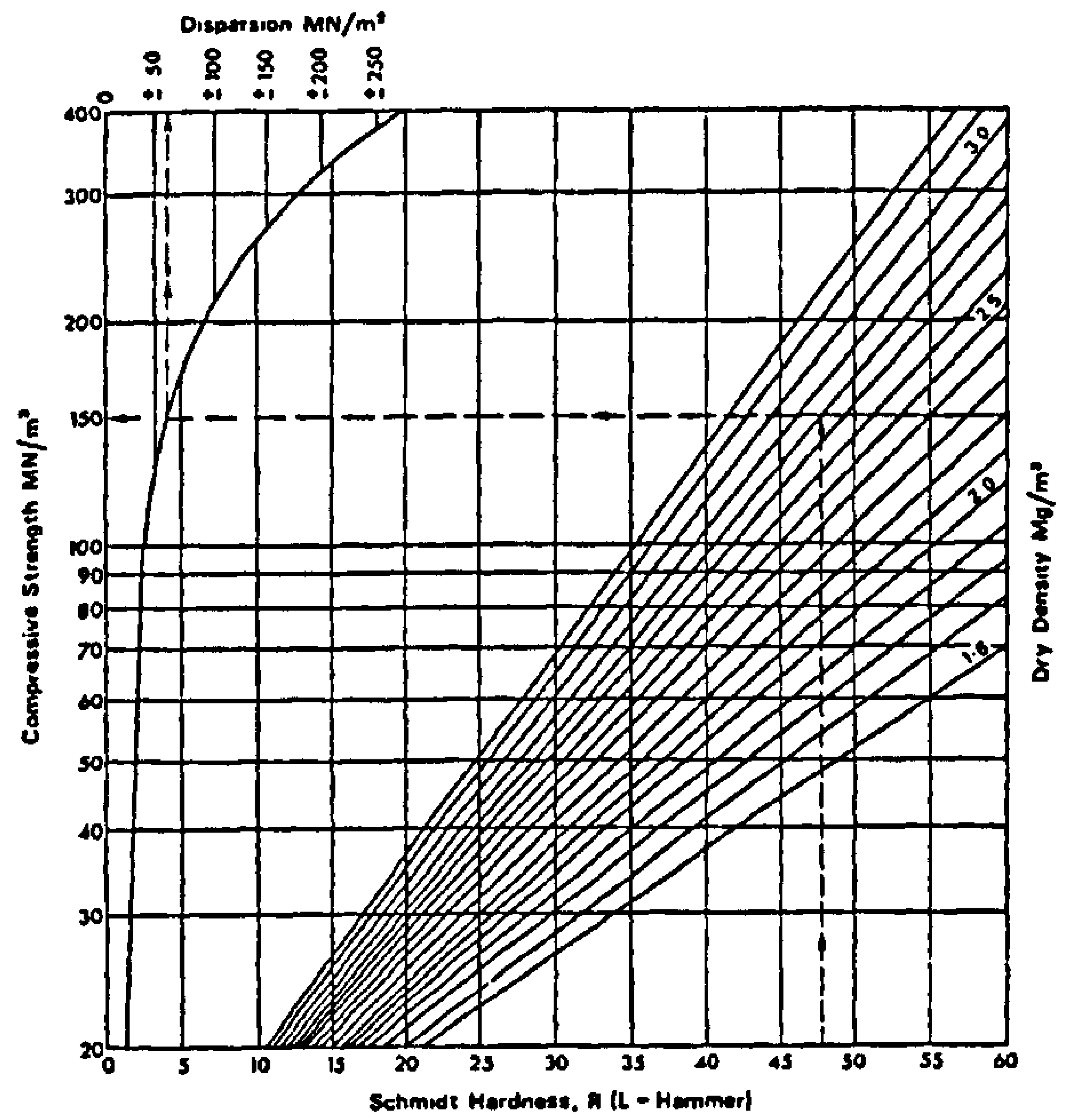


Figure 2.5 Conversion of L-type Schmidt hammer rebound value to compressive strength. The chart assumes hammer used vertically downwards (*after Deere and Miller 1966*).

The size and shape of constituent minerals, the nature of the cementing material and their inter-relationships determine rock texture. Texture, in turn, is a principle determinant of rock strength and elasticity, and thus is important in determining the resistance of rock to weathering by mechanical processes. Elasticity describes the linear ratio of stress to strain, in which a given stress is required to bring about a certain amount of strain in a material. The more resistant the material to deformation, the higher its elasticity and the more stress that can be applied without any resulting deformation. Beyond the elastic limit, very brittle materials can fail instantaneously. In less brittle materials or where the load is applied at a very slow rate, plastic, non-recoverable strain can take place. Elastic properties of a rock depend upon its stiffness, compressibility and density (New 1976), and are a good reflection of its compressive strength and hardness (Allison 1988). Cooks (1983) suggests that elasticity is one of the most influential factors in determining rock durability to weathering.

Good correlation has been established between the sonic velocity of rocks and elasticity and dry density, which in turn, are related to porosity and the percentage of sound minerals (Dearman et al 1987). That the sonic velocity of rock material is usually less than that of the fresh mineral constituents is an indication of weak cement bonds, fracturing, structural deformation, weathered and altered minerals, and the state of applied stress (New 1976; Whiteley 1983). Attempts to correlate variation in ultrasonic velocity with rock weathering susceptibility have been reasonably successful (eg Remy et al 1994).

The properties of constituent minerals, notably texture and the nature of grain contacts, also determine the thermal conductivity of the rock (eg McGreevy and Smith 1982; McGreevy 1985). Thermal conductivity has a direct bearing on susceptibility of rock to insolation weathering (Winkler 1994) and has implications for the efficacy of salt weathering (McGreevy and Smith

1982). The chemical composition of mineral constituents will play a significant role in determining the reactions which take place in the present of water and influence the generation and composition of percolating fluids due to chemical weathering processes.

2.2.3 Structural properties: pre-existing flaws

The constituent mineral grains and the void space between them largely determine the nature of rock material and its response to weathering. However, superimposed on these intrinsic properties there are also small scale structural features and discontinuities. These include material flaws and planes of weakness such as fine laminations, mineral or rock cleavage, and grain associated microcracks. There is a tendency in experimental rock weathering studies to treat samples as uniform and the potential effects of these heterogeneities have rarely been addressed. Some notable exceptions to this are studies by Douglas (1981); Smith and McGreevy (1983); and Douglas *et al.* (1994). For experimental work, it is attractive to utilise samples which are free from visible defects and flaws, and indeed this might be essential; to test the influence of variations in sample geometry, or to assess the effect of varying temperature and moisture conditions, for example. It can be equally important, however, to represent field conditions as closely as possible in order for the results to be more widely applicable. This is not only important for geomorphic studies of landform development, but also for practical purposes, such as the assessment of rockslope deterioration susceptibility and rock durability testing in the selection of construction stone.

In addition to microcracks and other micro flaws, pre-existing flaws are regarded in the broad sense as any macroscopic features (ie visible to the unaided naked eye) which introduce mechanical and lithological heterogeneity into the rock material. Thus cracks are included because of their obvious potential to create planes of weakness in rock, but variations in mineralogy and even colour are also included because they often correspond with changes in weathering susceptibility.

Pre-existing flaws might contribute to total porosity and have a significant influence on permeability. As such, they provide potential pathways for fluid migration and thus can enhance the potential for chemical weathering. They also provide voids in which crystallisation of ice or salts, for instance, could occur. Pre-existing flaws could contain weathered clay minerals, the presence of which increases the possibility of rock damage due to volumetric expansion on wetting, for instance. Flaws, whether open or closed, also reduce rock strength and elasticity, and thus the rock resistance to physical weathering. If oriented, such as might occur with rock cleavage or parallel laminations, discontinuities can render a rock highly anisotropic such that its weathering behaviour differs considerably depending on the relationship between the discontinuity orientation and an exposed rock surface. McGreevy and Whalley (1985) have also argued that enhanced frost damage occurs at crack locations because of concentrations in moisture compared to the moisture content of the intact material. Experimental studies by Matsuoka (1990a) have revealed enhanced frost splitting along fractures in shales; and in field observations, Douglas (1981) and Douglas *et al.* (1994) noted the fundamental importance of small scale discontinuities in the large scale weathering of basalt cliffs. The mechanisms by which these pre-existing flaws contribute to rock breakdown are considered in section 2.4 below.

2.3 The Weathering Environment

Discussion of the mechanisms involved in rock weathering follows below but it is useful here to identify and briefly describe those processes which operate primarily, though not exclusively, at the rock mass scale. (a) *Block, root and frost wedging* involve the wedging apart of two sides of a discontinuity by loose rock fragments, woody vegetation stems or ice. This mechanism requires the presence of a pre-existing, open fracture and hence operates at the rock mass scale. (b) *Rebound fracturing* due to stress release can involve large scale gravitational and tectonic stresses. These processes are dealt with more fully in Part Two of the thesis.

2.3.1 Mechanical rock weathering processes

Mechanical weathering processes result in the rupture of rock as a result of applied forces which locally exceed the tensile strength of the rock material. These forces can involve applied stress from volumetric expansion of fluids or minerals, expansive recovery due to stress relief, and fatigue failure due to repeated moisture and/or temperature fluctuations.

2.3.1.1 Mechanisms of frost shattering

Frost shattering has been described by Ollier (1969, p11) as:

> "....one of the greatest, if not the greatest mechanical agent in weathering"

Despite this, the lack of data, knowledge and understanding or frost shattering mechanisms is widely acknowledged (Ives 1973; McGreevy 1981; Lautridou 1988). Because of the very wide range of rock properties and environmental conditions which interplay with freeze-thaw, it has been difficult to isolate the key controlling factors in the process. For example, the permutations of environmental conditions reviewed by McGreevy (1982) include rock moisture content, freezing intensity and duration, and the number and amplitude of freezing cycles. The range of natural conditions under which freeze-thaw operates and the precise characterisation of those environments has made it difficult to model realistic moisture and temperature conditions in experimental work (White 1976). The lack of an adopted standard for moisture and temperature conditions has also made it difficult for results to be compared.

Attempts to correlate rock properties with freeze-thaw weathering have also met with variable success. For instance, investigations of the influence of rock type (Potts 1970; Keeble 1971) have been contradictory and studies of the role of porosity in frost susceptibility (Wiman 1963; Potts 1970; Keeble 1971) have been inconclusive. Understanding of frost shattering mechanisms is further complicated by the fact that the process is unlikely to act in isolation from chemical weathering processes (eg Keeble 1971). Nevertheless, several theories to explain freeze-thaw shattering of rock have been advanced and these are considered below.

Volumetric expansion of water on freezing: The volumetric expansion of water on freezing has the potential to apply a pressure of up to 207MPa at -22°C (Bridgman 1912) and this has conventionally been regarded as a likely mechanism for frost shattering of rocks (eg Ollier

1984). However, some problems with this hypothesis were identified by Grawe (1936). He argued that a system in which such high pressures could be achieved would need to be closed, and that if this were the case, by what mechanism could water enter the pore or crack structure in the first place? Battle (1960) suggested that a water-filled crack might freeze from the top down, effectively creating an ice 'seal', and thus a closed system. However, the creation of a seal and the generation of lateral stresses from the ice wedge depended, according to Battle, upon a very rapid freezing rate of 0.1°C min^{-1}. Without this, he envisaged the 'plug' could simply be expelled. A photoelastic study of ice pressure in cracks (Davidson and Nye 1985) indicated that expulsion of an ice seal is more likely to be influenced by the properties of the crack and its walls than the rate or intensity of freezing. Although a very rapid rate of freezing has correlated well with increasing frost damage in experimental work (Battle 1960; Mellor 1970), the rates used are unlikely to occur in nature. Grawe (1936) also argued that the potential for pressure to be applied on rock would be reduced in systems which were not fully saturated, because any air contained within the rock could be compressed. He also stated that except in extreme environments, temperatures of -22°C were unlikely to exist.

Consequently, it is by no means certain that even the most fundamental requirement of the volumetric expansion theory, a closed system, could be achieved. Walder and Hallett (1986) have added the observation that frost heave in soils commonly exceeds that which would occur if volumetric expansion of ice were the only process operating. This indicates that even if volumetric expansion could explain *some* frost shattering, other mechanisms must also be found.

A number of alternate hypotheses have been proposed and these are considered briefly. Full reviews have been undertaken by McGreevy (1981, 1982), Lautridou (1988), and Hall (1988b).

Direct pressure due to ice crystal growth: Evans (1970) proposed that direct pressure due to the growth of ice crystals, analogous to the forces exerted during salt crystallisation from solution, could be sufficient to cause rock fracture. Since a supply of water to the crystal growth face is needed for ice crystals to grow, an open system is required (McGreevy 1981). Migration of supercooled water to crystal growth faces via narrow microcracks and pores is likely to yield the greatest crystallisation pressure (McGreevy 1982). This theory for frost shattering appears to be compatible with the water migration theory outlined below.

Ordered water hypothesis: Experimental work by Dunn and Hudec (1966) on clay-rich and carbonate rocks forms the basis for the 'ordered water hypothesis'. Their observation that water can remain unfrozen at low temperatures (also Mellor 1970) is critical to the hypothesis. At low temperatures, water molecules adsorbed onto the surfaces of minerals become 'ordered', ie they become polarised and oriented. The dipole nature of water molecules means that one end of the molecule bonds to the mineral grains, while the other free end projects into the pore space. Where this pore space is small (<5μm diameter) similarly charged free ends might be in sufficiently close proximity as to repel each other. The resulting repulsive force is increased at lower temperatures and where total water adsorption is high, such as in clays and other minerals with a high surface area. The forces involved are thought to be sufficient to cause shattering of rock, though Dunn and Hudec (1966) themselves warned against applying the theory beyond clay-rich rocks. White (1976) suggested that the mechanism could explain rock breakdown in climates where freezing is absent or rare.

Hydraulic pressure: In freezing conditions, rock breakdown can be induced by hydraulic pressure, a theory advanced by Powers (1945) following experimental work on concrete. In a material where the surface layer is fully saturated and is in contact with freezing temperatures, the surface layer freezes first, forming a seal, and a freezing front migrates inwards. If the system is closed, water will be displaced inwards, and might create sufficiently high pressures as to cause rock shattering.

Water migration (capillary theory): The capillary theory of frost heave in soils and the concept of 'segregated ice' proposed by Taber (1929, 1930), has more recently been applied to frost shattering in rock (Walder and Hallett 1985, 1986; Tharp 1987), though Mellor (1970) also recognised the potential application of Taber's theory to rock. The water migration theory also owes much to the findings of Dunn and Hudec (1966, 1972) and others who have shown that water can remain unfrozen in rock, even at low temperatures. Following Walder and Hallett (1985), water migration due to adsorptive suction occurs because a thin film of water exists between the ice front and the rock substrate. This water has reduced chemical potential and attracts pore water from beyond the frozen fringe and towards the ice-water interface. This provides a supply of moisture for continual expansion of the ice body (Tharp 1987). This film of water creates a 'disjoining force' (Gilpin 1980) at the ice-rock interface and is the medium through which ice pressure is exerted to cause crack growth (Hall 1988b) (Figure 2.6).

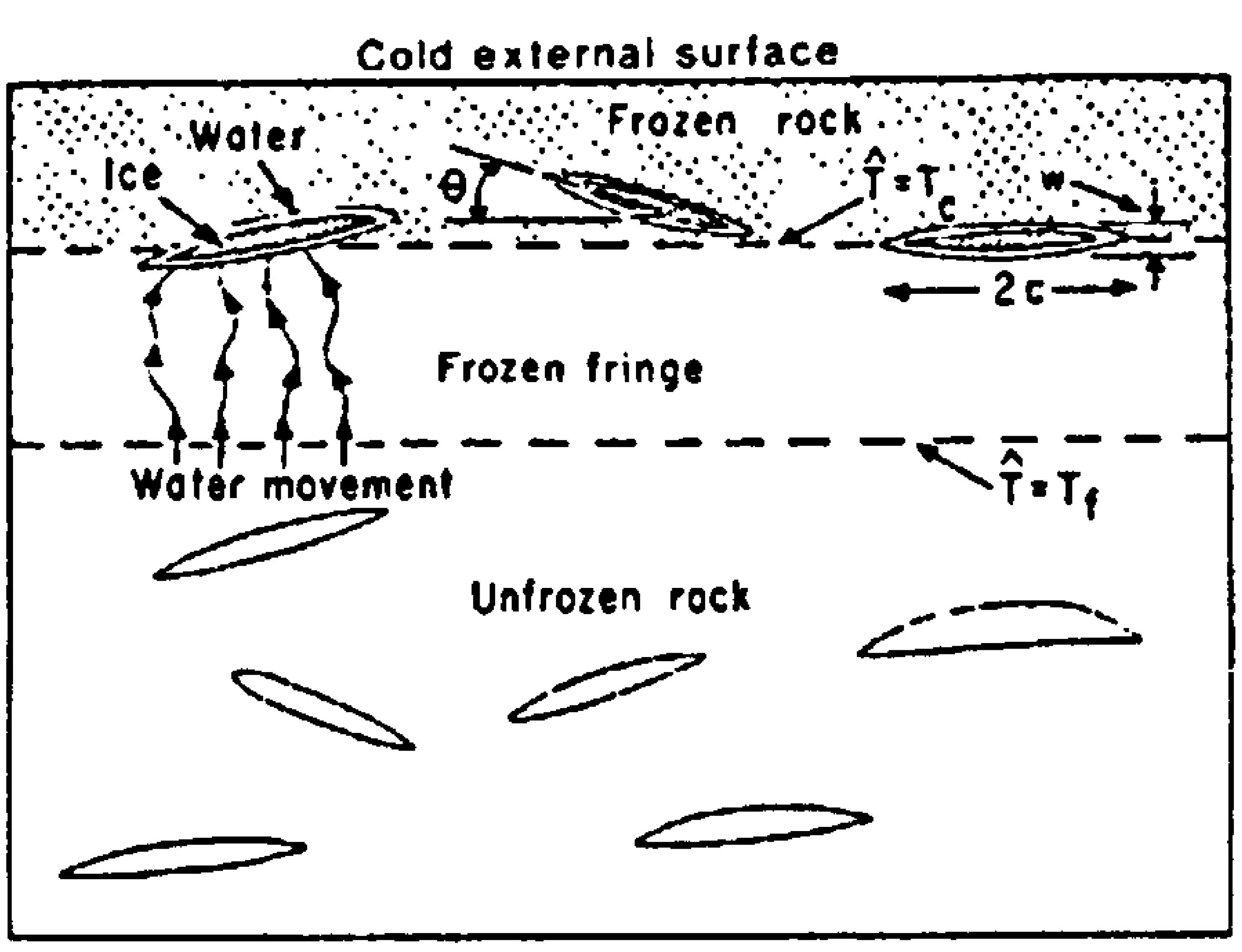


Figure 2.6 Frost weathering of cracked rock according to the water migration theory.

Cracks are penny shaped, of width *w* and diameter *2c*. The temperature relative to 0°C, $\hat{T}$, is T_c at the crack wall. Below the isotherm where $\hat{T}=T_f$ rock is unfrozen. Water migrates from the unfrozen rock, through the frozen fringe, and to cracks which are filled with ice (*after Walder and Hallett 1985*).

Walder and Hallett (1985) propose a mathematical model which predicts crack growth rates due to freezing under given conditions. The results are in accord with experimental findings and the conceptual work of Taber (1950).

This water migration theory requires an *open* system with abundant water supply at atmospheric pressure, and a *slow* rate of cooling. These conditions are likely to occur commonly in rock masses where uni-directional freezing occurs, such as on a rockslope. Where multi-directional freezing occurs, such as in the case of an isolated rock fragment, the freezing front can encroach from several surfaces creating a closed system in which water pressures might be sufficient to cause hydrofracturing (Walder and Hallett 1985) (the 'hydraulic pressure' hypothesis described above). This is a further indication that several mechanisms operating together might be responsible for frost cracking. Powers and Helmuth (1953, reported in McGreevy 1982) argued that both in situ volumetric expansion and the growth of segregated ice have a role in frost damage of building materials. They suggest that the role of water migration becomes more important with slower freezing rates and with increased duration of freezing. This accords well with Walder and Hallett (1985) who conclude that in open systems where there is no build up of water pressure, slow cooling rates of around 0.1-0.5°C h^{-1} and prolonged freezing, favour frost cracking. They also conclude that fluctuation of temperatures about 0°C is not necessary for crack growth and that prolonged crack wall temperatures of -4°C to -15°C provide optimum conditions for crack growth in the rocks investigated. These findings contradict the traditionally regarded importance of the *number* of cycles of freezing and thawing about 0°C and places more emphasis on duration of freezing.

Matsuoka (1990a) attempted to verify the relative roles of different frost shattering mechanisms and found in a saturated, open system that volumetric expansion participated in frost shattering for some rock types, but the primary mechanism was water migration by adsorptive suction. Matsuoka also undertook experiments under varying degrees of saturation in a closed system and found that significant increase in sample deterioration occurred where the saturation coefficient (S) exceeded 0.75. This value accords well with other investigations (Hirschwald 1912; Tourenq 1970) in which the concept of a threshold degree of saturation is recognised. It has been argued that volumetric expansion should not occur where S is less than 0.92 since ice pressure would simply be taken up by extrusion into air-filled voids (McGreevy and Whalley 1985; Matsuoka 1990a). This figure of 0.75 is used as evidence by Matsuoka (1990a) to conclude that volumetric expansion cannot be the only mechanism causing breakdown. However, this argument is simplistic because S simply represents the average moisture content of the rock. In reality, moisture is unlikely to be uniformly distributed within the rock and moisture gradients probably exist (McGreevy and Whalley 1985).

Nevertheless, Matsuoka (1990a) demonstrated that for samples under identical saturation conditions, frost shattering was an order of magnitude greater in open systems with a constant moisture supply, than in closed, finite water supply systems. In frost shattering, the importance of moisture supply at the ice front cannot be understated; if water migration is the principle mechanism by which frost shattering occurs, then perhaps degree of saturation should be considered less in terms of a critical threshold (eg McGreevy and Whalley 1985) and more in terms of an indication of moisture influx capacity. The likelihood is that more than one mechanism is responsible for frost shattering (eg Cady 1969, Konischev 1978), and that the processes described above are not mutually exclusive, but operate in combination.

The role of variations in *environmental conditions* is critical in any understanding of frost shattering mechanisms and these are reviewed by McGreevy and Whalley (1985). Granular disintegration and the accumulation of angular screes at the foot of cliffs have traditionally been attributed to freeze-thaw, although other forms of breakdown, including flaking, scaling and in situ fracturing and wedging have also been recognised. However, recent work indicates the importance of other mechanisms operating in association with freeze-thaw mechanisms such as wetting and drying (Bland and Rolls 1998), salt weathering (Jerwood et al 1987, 199a, 1990b) and chemical processes (Whalley et al 1982; Whalley et al 2000). Consequently, it is necessary to re-asses the nature of rock breakdown due to freeze-thaw and to question these traditional assumptions.

Certain *rock properties* are repeatedly recognised as being of considerable importance in frost weathering. These include pore structure (McGreevy 1981, 1982; Lautridou and Ozouf 1982; Lautridou 1988), saturation coefficient (McGreevy and Whalley 1985; Hall 1988a; Matsuoka 1990b), water absorption capacity (McGreevy 1982; Lautridou 1988); mechanical strength (Lautridou and Ozouf 1982; Matsuoka 1990a) and microporosity (McGreevy 1982). Porosity has also been correlated with frost susceptibility (Lautridou and Ozouf 1982; Lautridou 1988) though Matsuoka (1990a) suggested these relations do not hold for igneous rocks. Matsuoka (1990a) also found an inverse relationship between frost susceptibility and tensile strength. Other rock properties have also been cited, including the influence of rock anisotropy on freezing penetration (Hall 1986b), the state of weathering (Brockie 1972), and crack properties and geometry (Davidson and Nye 1985; Lautridou et al 1986; Tharp 1987). These findings have strongly influenced the planning of the experimental work in this thesis.

2.3.1.2 Mechanisms of salt weathering

Salt weathering is mostly associated with arid regions (eg Cooke and Warren 1973) where evaporation rates are high, and with coastal zones where the supply of salt from sea spray is plentiful (Winkler 1994; Moses and Smith 1994; Mottershead 1994). However, significant salt weathering also occurs in inland temperate areas (eg Winkler 1994; Robinson and Williams 1996) where salts are derived from road de-icing, stone cleaning operations (Pombo-Fernandez 1999) and naturally, within pore and mineral structures. Salts derived from mortar, cement and de-icing leachate can also combine with rising groundwater. Further salts can be derived from atmospheric and groundwater pollution and from the natural weathering of pyrites (Winkler 1994).

Three principal mechanisms for salt weathering have been identified (after Cooke and Smalley 1968): (i) *Thermal expansion and differential expansion.* Upon heating, salts can expand more than the surrounding rock, leading to rock rupture. A rock surface temperature rise of 40-50°C is required (Cooke and Smalley 1968) and thus this mechanism is favoured in arid climates. (ii) *Hydration pressure.* Absorption of water into the crystal lattice of salts increases crystal volume and exerts pressure against the pore walls. Hydrates are a more stable form of many types of salt and form from hydration in response to changes in both temperature and atmospheric humidity. Low temperatures and high relative humidity produce the greatest hydration pressures (Winkler and Willhelm 1970). For this mechanism to be effective, hydration pressure must exceed that of the tensile strength of the rock. Hydration pressures can reach 100MPa (Ollier 1984) and even more in some cases (Winkler and Willhelm 1970). (iii) *Crystallisation pressure.* When salts crystallise from a supersaturated solution a volumetric expansion occurs capable of causing damage (Evans 1970). The pressure of salt growth is transmitted through a thin film of supersaturated solution (Winkler 1994). This mechanism is probably the most important salt weathering process (Cooke and Warren 1973). For a given concentration of salts in solution, a reduction in temperature increases the likelihood of crystallisation, and pressures reached are directly proportional to salt concentration (Winkler 1994). Although in theory, high crystallisation pressures can be generated, actual pressures are a function of the degree of supersaturation, temperature conditions, the extent to which the pore system is closed, the type of salts present (eg Goudie 1974; Williams and Robinson 1998) (Figure 2.7) and crystal properties including the rate of crystal growth. Given supersaturation by a factor of two, which can be achieved by evaporation (Cooke and Warren 1973), most of the commonly occurring salts such as halite, gypsum and anhydrite can generate pressures sufficient to damage rocks (Winkler 1994).

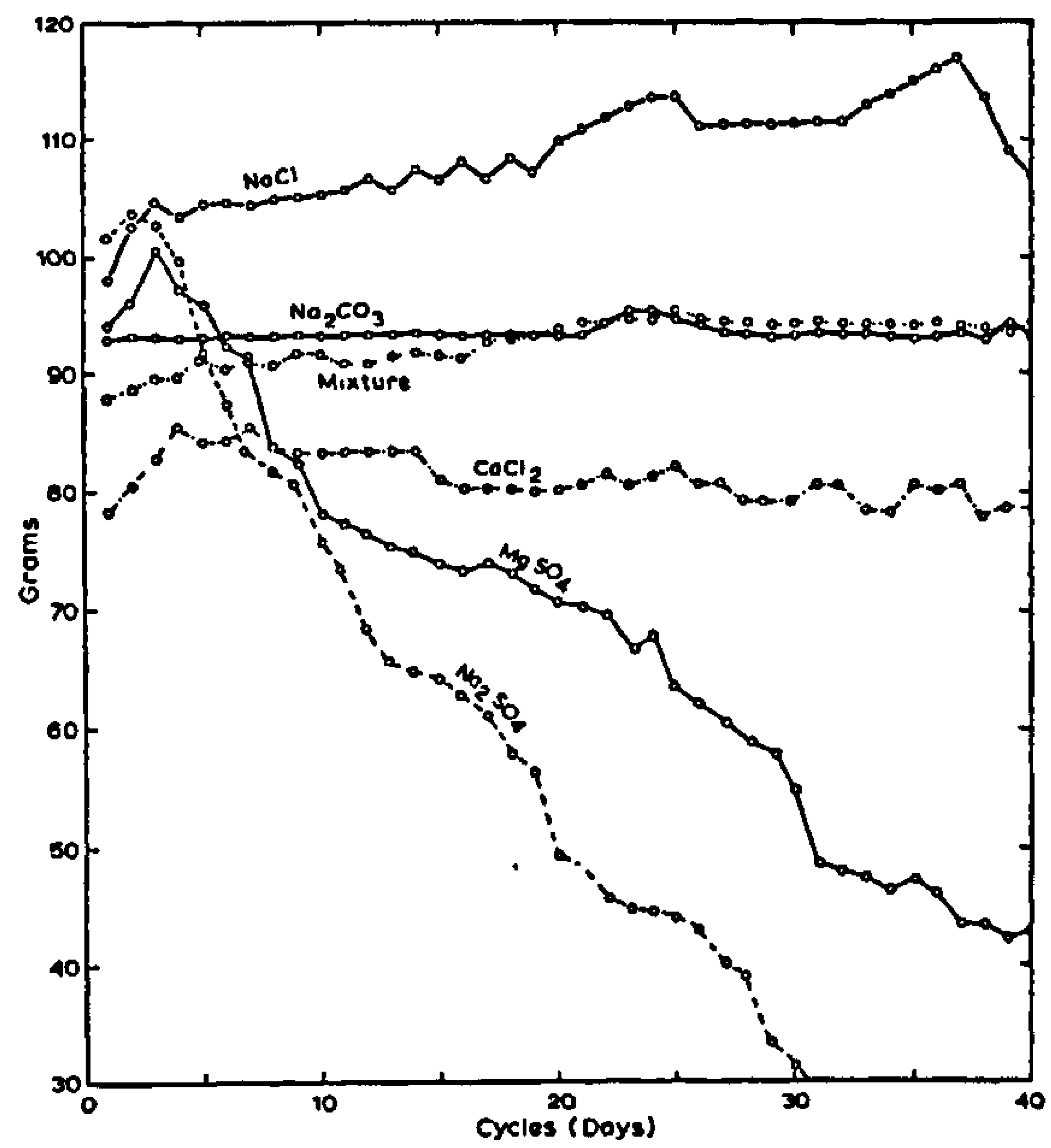


Figure 2.7 Variations in weight loss for sandstone due to treatment with different salts (*after Goudie et al 1970*)

In terms of rock properties important in salt weathering, the uptake of moisture in rock pores and cracks by capillary rise is critical (Cooke and Warren 1973). The structure and connectivity of the pore system also determines spatial distribution and penetration of damage zones. Work by Fitzner (1988) suggests that salt solutions are more likely to fill macropores completely before the smallest pores are filled and this has implications for the potential role of pore structure in salt weathering. Other rock properties including total pore volume (Ginell 1994), clay mineralogy (McGreevy 1982), saturation coefficient (Honeyborne and Harris 1958), permeability and moisture absorption capacity (Cooke 1979; McGreevy 1996), and porosity and microporosity (Cooke 1979) have also been related to the efficacy of salt weathering. Other factors such as chemical effects of salt (eg dissolution) might also be relevant. Salt weathering does not generally create distinctive landforms, but leads to a range of deterioration modes including granular disintegration, small scale exfoliation and flaking. It is also possible that salt weathering plays a role in honeycomb weathering (Mustoe 1982; Robinson and Williams 1994).

2.3.1.3 Mechanisms of wetting and drying

Experimental work (eg Goudie et al 1970; Ollier 1984, who reports on unpublished work by Condon) has shown that rocks subjected to repeated cycles of wetting and drying are prone to disintegration. Rocks with an absence of clay minerals appear to be much more resistant to wetting and drying weathering (Bland and Rolls 1998), but their *presence* does not necessarily increase susceptibility. This was shown in work by McGreevy (1982) where among a variety of rocks tested including limestones, chalks and sandstones, most containing clay minerals, a basalt was the only rock affected by repeated cycles of wetting and drying. Swelling expansion associated with wetting and drying has also been observed in granites and sandstones (Hockman and Kessler 1950).

Clay minerals (eg smectite) can disrupt the rock material by swelling or by operation of the ordered water process described above (section 2.2.1.1). The repulsive forces set up in pore spaces due to the polarisation and orientation of water molecules are removed when the rock is dried by evaporation. Thus as the rock is repeatedly wetted and dried, molecular forces bring about cyclic expansion and contraction of pore spaces, leading to damage (Cooke and Warren 1973; Bland and Rolls 1998). Non-swelling clay minerals such as illite and kaolinite are often involved in this mechanism. It is also thought that the susceptibility of rocks to wetting and drying weathering is enhanced by the presence of micro-structural weaknesses which allow water ingress into the rock (Olivier 1979b), or by low temperatures.

Studies of mudrocks (Olivier 1979a, 1979b) have shown that in addition to alternating wet and dry episodes, cyclic fluctuations in humidity also lead to disintegration. Cyclic swelling and contraction due to humidity changes can produce measurable strains in rock sufficient to produce extensive fracturing (Cummings 1987). However, even at 98% relative humidity, the effects are not as severe as when the rock is completely submerged (Olivier 1979b).

Selby (1993) also describes 'air breakage fracture' which can occur when wetting occurs very rapidly after a dry period with significant evaporation. This creates a suction effect between pores, trapping air which becomes compressed by capillary pressures. This pressure can be sufficient for rock fracture to occur.

Deterioration of rocks attributed to cyclic processes such as freeze-thaw and salt weathering, might in part, be due to the alternate wetting and drying which usually accompanies these processes (Bland and Rolls 1998).

2.3.1.4 Mechanisms of slaking

The slake durability test is an index test, the main purpose of which is to "evaluate the weathering resistance of shales, mudstones, siltstones and other clay-bearing rocks" (Franklin and Chandra 1972, p325). The latter include chalk, sandstone and weathered igneous rocks. The test was devised as an improvement to the widely used practice of immersing rocks in water to observe signs of swelling or disintegration, often regarded as an indication of susceptibility to wetting and drying. Slake durability is primarily influenced by rock properties which allow ingress of water into the rock material and those which influence the behaviour of fluids once in the rock (ie permeability and porosity). The presence of clay minerals enhances rock susceptibility to slaking (Franklin et al 1971), probably a reflection of the role of water adsorption or swelling. Dissolution of grain contacts is determined by mineral grain and matrix composition. The ability of the rock to resist disruption also influences slake durability (Franklin and Chandra 1972).

Despite efforts in the test design to minimise attrition and agitation (Franklin and Chandra 1972) the test inevitably involves some rock-on-rock abrasion and impact. The contribution of these mechanisms to sample disaggregation compared to that which results directly from slaking is unknown. However, the contribution to apparent slake durability of abrasion and impact during the test should not be underestimated.

2.3.1.5 Thermal weathering (insolation)

Rock temperature fluctuations can lead to expansion upon heating and contraction upon cooling, sufficient to cause rock fracture by fatigue. This can occur because rock is a poor conductor of heat, so thermal gradients are established from the surface to the interior of the rock mass. Expansion and contraction due to temperature fluctuations occurs differentially. Further stresses can develop due to differential expansion and contraction of adjacent minerals with contrasting thermal properties. Rock stresses thus developed might be sufficient to cause microfractures and granular disintegration. Cooke and Smalley (1968) measured the thermal expansion of several salts and found most to have a volumetric expansion greater than that found in a typical granite. This differential expansion creates stress concentrations sufficient to lead to crack propagation (Selby 1993).

The amount of solar insolation received is affected by external, fixed factors such as aspect, latitude, altitude and angle of incidence; climatic factors such as wind speed, cloud cover and air temperature; and rock properties such as surface albedo (the ratio of reflected radiation to total incident radiation), thermal conductivity and coefficient of thermal expansion (Bland and Rolls 1998). Early experimental work suggested that insolation is much less effective in the absence of moisture (Blackwelder 1925; Griggs 1936). This might be because the expansion of water upon heating can be much greater than the equivalent rock expansion. If the water is confined, then substantial pressures can be generated in rock pores (Winkler 1994). This would also explain how insolation weathering is more effective under conditions of partial confinement (Selby 1993). However, recent work by Hall (2000) suggests that thermal shock can operate in the absence of water where sharp temperature changes $>2^{\circ}C\ min^{-1}$ occur (Yatsu 1988). This rate of temperature change has been recorded in arid environments in the Antarctic (Hall 2000) and southern Africa (Meiklejohn 2000), notably as a result of broken cloud movement on very hot days. This development challenges the concept that insolation weathering relates to diurnal or seasonal fluctuations in temperature. The indications are that large stresses can be generated by much more rapid fluctuations, perhaps occurring many times per day.

Insolation weathering is characterised by exfoliation and blocky disintegration of the rock mass.

2.3.1.6 Other mechanical weathering processes

Falling or sliding rock fragments can contribute to direct fracture, detachment and removal of in situ rock by the effects of abrasion and impact. Similar effects can arise from human and animal disturbance. The contribution of this process to rockslope deterioration is difficult to quantify but is likely to be small and localised.

2.3.2 Chemical weathering

Chemical weathering generally occurs in the presence of water and as the result of the migration of fluids through void spaces in rock. The amount and nature of void space and its inter-connectivity therefore play a considerable role in many chemical weathering processes. Even where mechanisms operate primarily at the rock surface, moisture from atmospheric humidity, dew and precipitation is usually an essential component. Although there is a range of mechanisms involved in chemical weathering, three net effects can be identified: minerals can be dissolved, new minerals can form (or existing minerals be substantially modified), and residual minerals can be released. If, as a result of these processes, an increase in mineral volume occurs, then the mechanical forces exerted from volumetric expansion can be sufficient to cause rock damage (Price 1995).

2.3.2.1 Dissolution

Dissolution is usually the first stage of chemical weathering (Ollier 1969) and operates where water in contact with a mineral acts as a solvent. The amount of dissolution which can occur partly depends upon the solubility of the mineral. The general order of solubility for common rock-forming minerals is as follows (Polynov 1937):

Ca > Na > Mg > K > Si > Al > Fe

However, solubility is also dependent on pH value. Price (1995) provides the example that Fe is around 100,000 times more soluble in contact with a solvent with pH6 than pH8.5 (Figure 2.8).

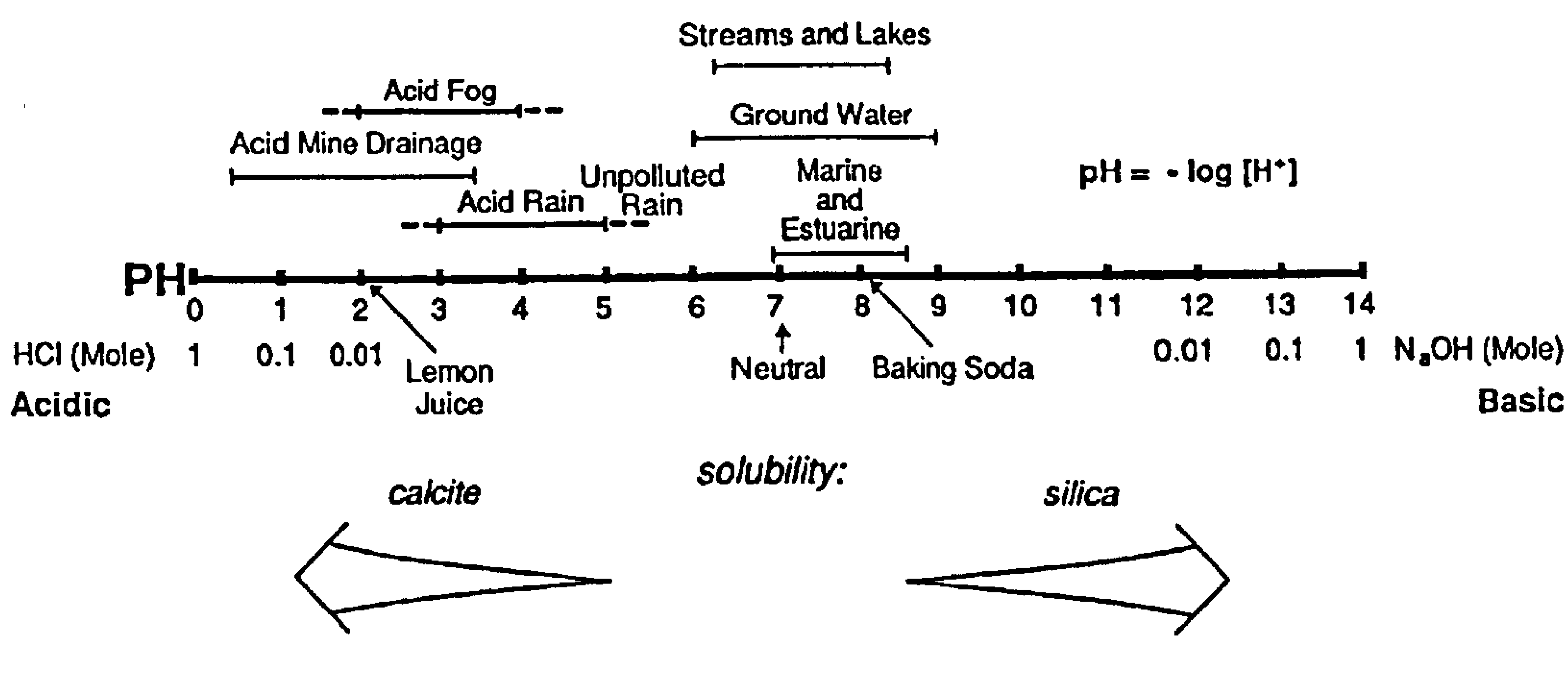


Figure 2.8 The relationship between pH, solvent type and mineral solubility (*after Winkler 1994*).

Dissolution is also partly controlled by aggressivity of the solvent. As more mineral matter is taken up into solution it approaches saturation and the capacity for further dissolution is reduced. At supersaturation levels, minerals can be precipitated. Other factors which influence dissolution are movement of the solvent (Winkler 1994), contact time between the solvent and the mineral (Brunsden 1979a), temperature and pH of the solvent (Trombe 1952; Picknett 1977) and the effect of mixing corrosion (Bogli 1971).

In a temperate environment, distinctive landforms can result from the dissolution of limestone, producing karst terrain which is characterised by an absence of surface runoff, sinkholes, underground cavities and cave systems, and a wide variety of meso and micro solution forms. Limestone solution is particularly sensitive to the carbon dioxide content of the solvent, which can be increased above atmospheric equilibrium by inputs from soil air rich in CO_2 from organic activity.

Rates of dissolution have been determined using a wide variety of techniques (Gunn 1981; Crowther 1983; Livingston and Baer 1988; Trudgill et al 1990; Pentecost 1991; Mottershead 1994; Inkpen 1998; Robinson and Williams 1998; Williams and Robinson 2000) and reviews of techniques are given in Whalley and McGreevy (1988) and Winkler (1994).

Stylolites are a peculiar feature of pressure solution occurring in hard limestones and chalks. Stylolites form highly irregular surfaces, often containing thin layers of clay which are vulnerable to weathering processes (eg McGreevy 1982; Winkler 1989).

2.3.2.2 Hydration

When water is added to a mineral it can become adsorbed into the crystal lattice forming new minerals and rendering them more porous and vulnerable to further weathering. For instance, iron oxides can be modified to hydrated iron oxides or iron hydroxides (Ollier 1969; Selby 1993). Hydration is also a very important breakdown mechanism in rocks containing clay minerals since they can undergo considerable volumetric expansion. This expansion is thought to be largely responsible for the rock breakdown which accompanies slaking and for granular disintegration and exfoliation of rocks in general (Ollier 1969). The hydration shattering described by White (1976) is the same mechanism and can equally be regarded as a physical weathering process. Brunsden (1979a) notes two further potential effects of hydration. First, salts present in the crystal lattice are subject to hydration when water is absorbed, enabling salt expansion and the disruption which might accompany this. Second, absorption of water at this molecular level promotes other water-based processes such as hydrolysis and dissolution.

2.3.2.3 Hydrolysis

Hydrolysis is a chemical reaction which takes place between mineral cations and the H^+ and OH^- ions in water. Hydrolysis can take place whenever minerals and water are in contact, even if the water is neutral, and water can act as a reactant as well as a solvent. Hydrolysis changes the concentration of hydrogen ions (ie the pH) and this influences the solubility of certain minerals, especially silica. Coarse grained feldspar-rich rocks are known to be very susceptible to this process (Brunsden 1979a), which can also result in expansion and contraction of silicate crystal structures leading to physical damage. A common example of hydrolysis is the production of kaolinite clays from the decomposition of orthoclase feldspar.

2.3.2.4 Oxidation, reduction and chelation

For the purposes of this account, minerals become oxidised in the presence of oxygenated water, producing new, stable minerals, of which iron oxides and hydroxides are the most common. Oxidation is favoured by aerobic, high temperature environments where organic matter is being destroyed. Mineral oxidation is accompanied by a colour change to red and/or yellow which dominate the host rock even when only present as a thin film coating around grains (Smith 1999). Oxidation also results in acid solutions being generated which enhances other chemical processes such as dissolution (Geological Society Engineering Group Working Party 1995).

Reduction is the reverse of oxidation and occurs in anaerobic environments where oxygen is depleted, such as permanently saturated conditions. Low temperature, high moisture environments favour reduction, which is enhanced in the presence of reducing agents such as organic matter. The potential for oxidation and reduction is also determined by the acidity of percolating water and the carbon dioxide content of air contained within it.

Chelation is a term which describes the uptake of metal ions into an organic ring structure. Chelating agents contained in plants are able to extract ions or nutrients for plant growth and decomposition of organic compounds can release chelating agents. This means that ion exchange occurs where it would normally not, and thus enhanced rates of mineral weathering are achieved. Acid organic solutions are also produced and can contribute to other chemical effects.

Chemical weathering can lead to the in situ disintegration and decomposition of large volumes of rock, rendering the material soil-like, though its properties will differ significantly from those of a transported soil (Geological Society Engineering Group Working Party 1995).

2.4 Rock Deterioration: Incipient Fracturing

The influence of rock flaws on breakdown at the scale of *microcracks* has long been recognised in classic fracture mechanics theory (Griffith 1920). In principle, the rupture of rock, potentially leading to general breakdown, depends upon the concentration of tensile stresses at flaws. However, before examining the mechanisms of physical rock breakdown, some definition of terms and discussion of concepts is offered.

2.4.1 Fractures in rock: definitions and terminology

In this thesis the term *discontinuity* is used in a broader context than provided by the ISRM definition (1978b) as an umbrella term to cover all types of mechanical break *and* structural boundary. The definition of Hencher (1987, p149) is adopted:

> "a boundary or break within the soil or rock mass which marks a change in engineering characteristics or which itself results in a change in the mass properties".

Thus a discontinuity includes mechanical breaks with zero tensile strength and structural boundaries, such as bedding planes, contacts and veins (Cruden 1975). These might not result in any modification to tensile strength, although there might be changes in other rock properties. A similar definition is adopted by Aydan and Kawamoto (1990) and Hudson and Priest (1979). The term *fracture* is used to describe any discontinuity which has a tensile strength less than that of the intact rock, so some loss of cohesion has taken place (Dennis 1967). Fractures have a high aspect ratio, their length being much greater than their width (Chernyshev and Dearman 1991), so this definition would exclude some voids in rock such as intergranular pores and solution cavities. This definition moves away from the concept of *rupture* (eg Dennis 1967) which has genetic connotations, and thus allows the inclusion of lithological features such as bedding planes which have become separated.

It is clear that the amount of separation and thus the tensile strength between fracture walls can vary and in this respect, Kirnig's (1990) 'life cycle' approach is a useful concept. Fractures which retain some tensile strength can be regarded as 'incipient' (ie at the early stage of development). These include fractures which have an extremely small aperture such that some asperities are still in contact, or fractures which retain a significant amount of rock bridging. Hencher (pers com) usefully also distinguishes between *strong incipient fractures*, requiring the equivalent of a heavy hammer blow to break apart, and *weak incipient fractures*, which break apart under moderate hand pressure. Open fractures are those with little or no tensile strength and include fractures infilled with a low tensile strength material. At the late stage in the 'lifecycle', deposition of cementing material from percolating waters could lead to mineral infilling, producing 'healed' fractures. Hencher (pers com) also identifies *hidden* fractures where pore or microcrack alignment at a microscopic scale produces a 'potential' fracture, but which is invisible in hand specimen (Figure 2.9).

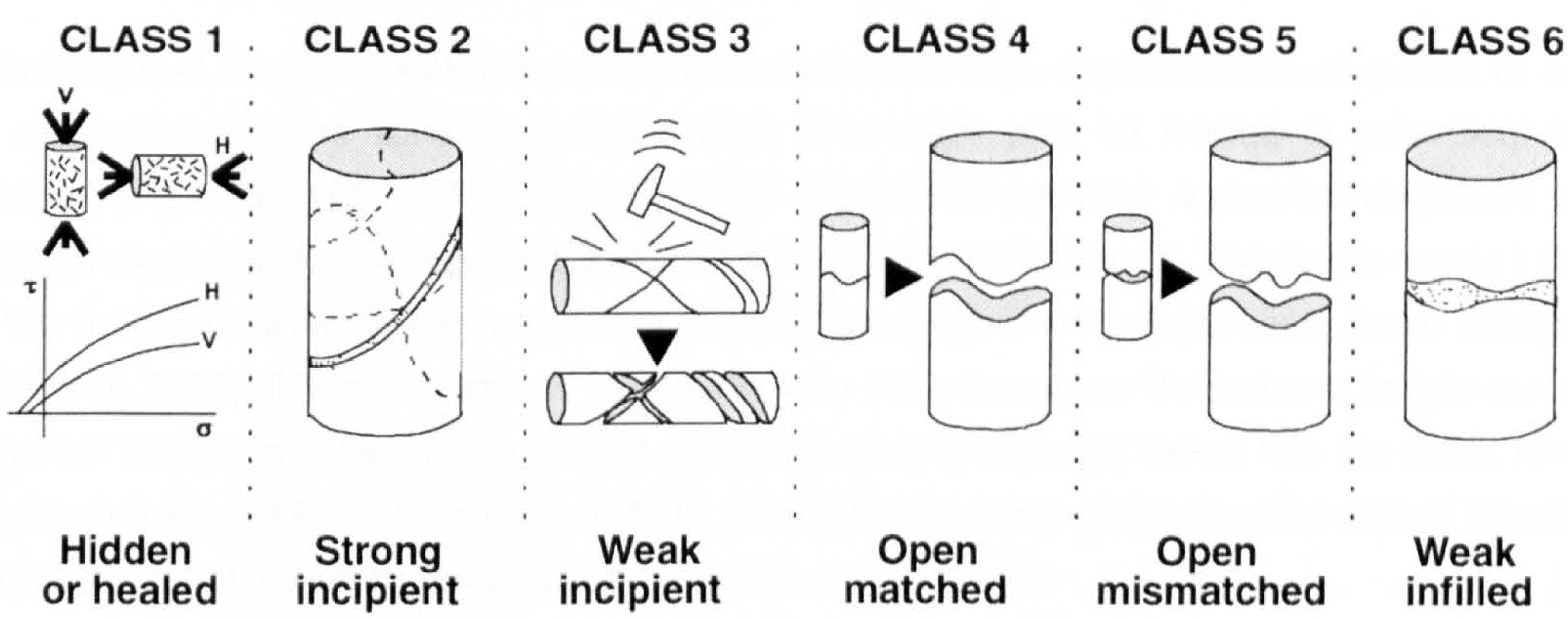


Figure 2.9 A 'lifecycle' approach to the classification of fractures (*based on Kirnig 1990, Hencher pers com, and Nicholson and Hencher 1996*)

The term 'fracture' does not suggest any mode of origin or scale and thus major tectonic joints, grain boundary microcracks and shear planes are all included. Definitions for different types of rock mass discontinuities follow those of Hencher (1987). *Tectonic joints* are "the result of orogenic stress in the earth's crust" and include fractures associated with folding and faulting; *faults* are "fractures along which displacement has occurred"; *bedding planes* are either "a change in sediment type or a hiatus in deposition" and often open up to form bedding plane joints; *cooling joints* form in igneous rocks as a result of contraction on cooling of magma; *rebound* or *sheeting joints* result from unloading (discussed further in Chapter Six). Metamorphic fabrics such as *slaty cleavage, schistocity, banding* and *foliation* can also be regarded as discontinuities. The term *fissure* is not clearly defined in the literature but is primarily used in the context of non-persistent fractures in unlithified clays and silts (eg Fookes and Wilson 1966; Fookes and Denness 1969) and other soft rocks and chalk (eg Fookes and Denness 1969). Other fractures can develop from desiccation, uplift, intrusion, erosion and weathering activity.

Fractures which are present at the micro scale are variously termed microcracks, cracks and microfractures (Kranz 1983). Simmons and Richter (1976) usefully distinguish between cracks associated with grain boundaries (grain boundary cracks), cracks which are wholly contained within grains (intragranular cracks) and cracks which cross two or more grains (intergranular or transgranular cracks respectively). Microcracks are not usually visible in hand specimen and can be up to 100 or 1000μm (Kranz 1983; Engelder 1987). Simmons and Richter (1976) identified different types of microcrack and their genesis. Cracks induced by local stresses and by thermal cycling are the types of microcrack of particular relevance to this research.

2.4.2 Concepts in fracture mechanics

Cracks can be generated in rocks where local stress exceeds local strength (Hoek 1968; Simmons and Richter 1976). This is known as the *strength of materials approach* to crack initiation and propagation (Ingraffea 1987) and assumes elastic-brittle behaviour where once local rock tensile strength is exceeded, instantaneous failure occurs. However, observations of experimental tests have shown that initiation is often not instantaneous and that behaviour after peak stress can be more complicated (Labuz et al 1985) (Figure 2.10).

It is likely that this post peak plastic behaviour relates to time dependent development of cracks in a growing process zone (Ingraffea 1987). This might also be related to subcritical stress corrosion (Costin 1987). The *linear elastic fracture mechanics approach* considers crack initiation and growth with regard to a crack tip stress intensity factor K_I (relating to mode I growth in this instance), which is the magnitude of crack tip stress in a homogeneous, linear elastic rock (Atkinson 1987). There is a critical stress intensity (K_{Ic}), known as the fracture toughness index (eg Hall 1986a), at which crack growth will occur, and at stresses below this the crack remains stable (Ingraffea 1987). Stress intensity factor (K) is a function of the applied normal stress and the geometry of the crack, and can be found from:

$$K = Y\sigma r(\pi c)^{0.5} \qquad [2.2]$$

Where Y is a numerical factor to account for crack geometry, loading conditions and edge effects, σr is the remote applied stress, and c, for penny-shaped, internal cracks is half the crack length. Fracture toughness varies with direction of measurement in anisotropic rocks and correlates well with P-wave velocity (Obara et al 1992). McGreevy and Whalley (1985) have argued that enhanced frost damage occurs at crack tips because of concentrations in moisture compared to the moisture content of intact rock.

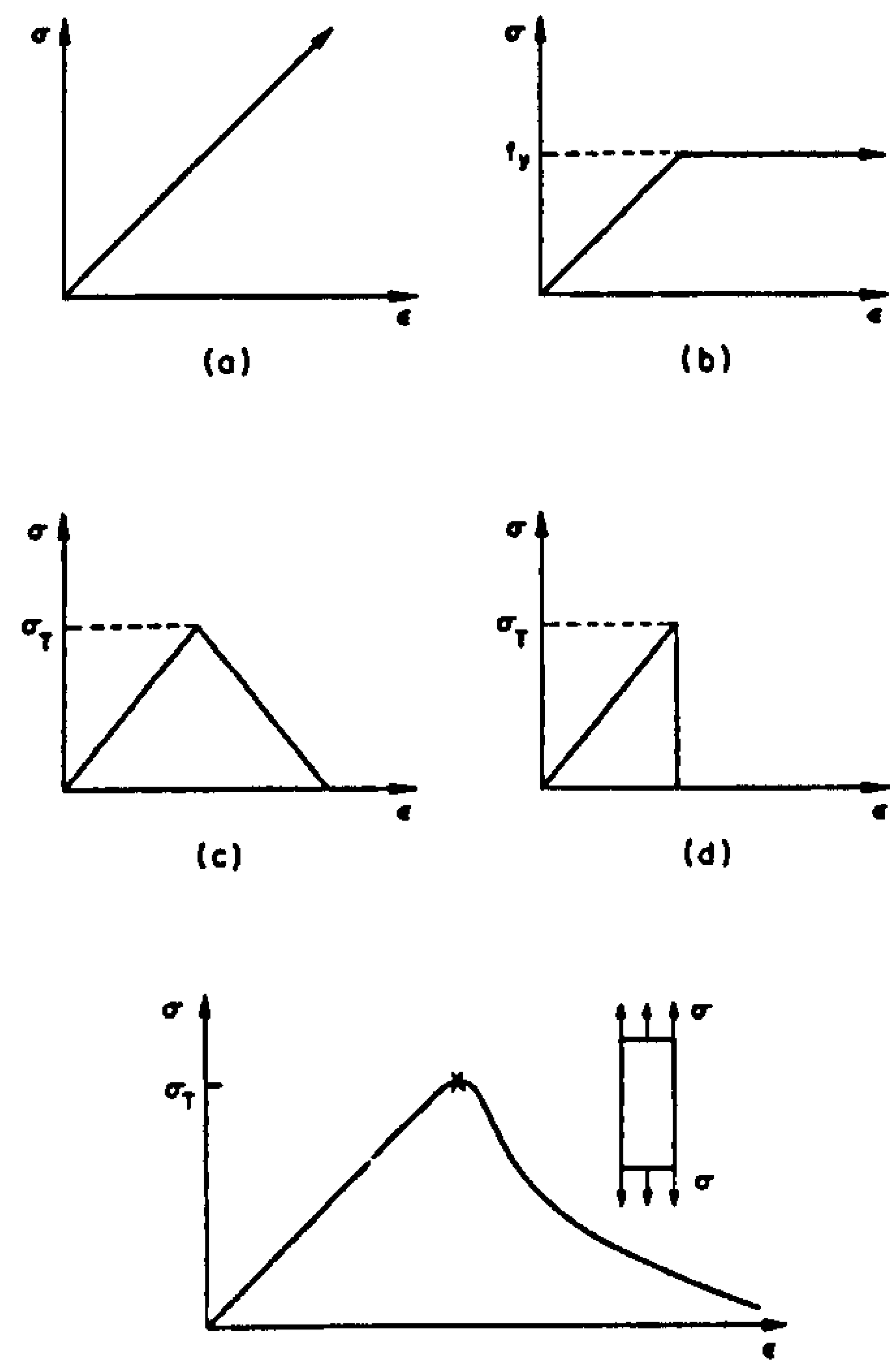


Figure 2.10 Idealisations of stress strain behaviour. (a) is a purely elastic material; (b) typifies the elastic-ductile behaviour of metals; (d) represents the way that rock is modelled – ie at peak stress instantaneous failure occurs. (c) and (e) show modelled and actual stress strain behaviour under uniaxial tensile loading, with time dependent deformation occurring in response to damage zone extension (*after Ingraffea 1987*).

The circumstances under which local tension exceeds local strength usually occur at flaws in rock (Griffith 1920; Lajtai 1977). These flaws are often modelled as microcracks, but can be pores or other flaws including cavities, fossils, grain boundaries, clasts and any features with properties which contrast with that of the host material or which create a boundary (Atkinson 1987; Pollard and Aydin 1988). Pollard and Aydin (1988) present idealisations of stress intensity for a range of different flaw geometries under different conditions of applied remote compressive stress and local tensile stress (Figure 2.11). A large collection of stress intensity factors has also been produced in the *Stress Intensity Factors Handbook* (Murakami 1987).

2.4.3 Subcritical crack growth

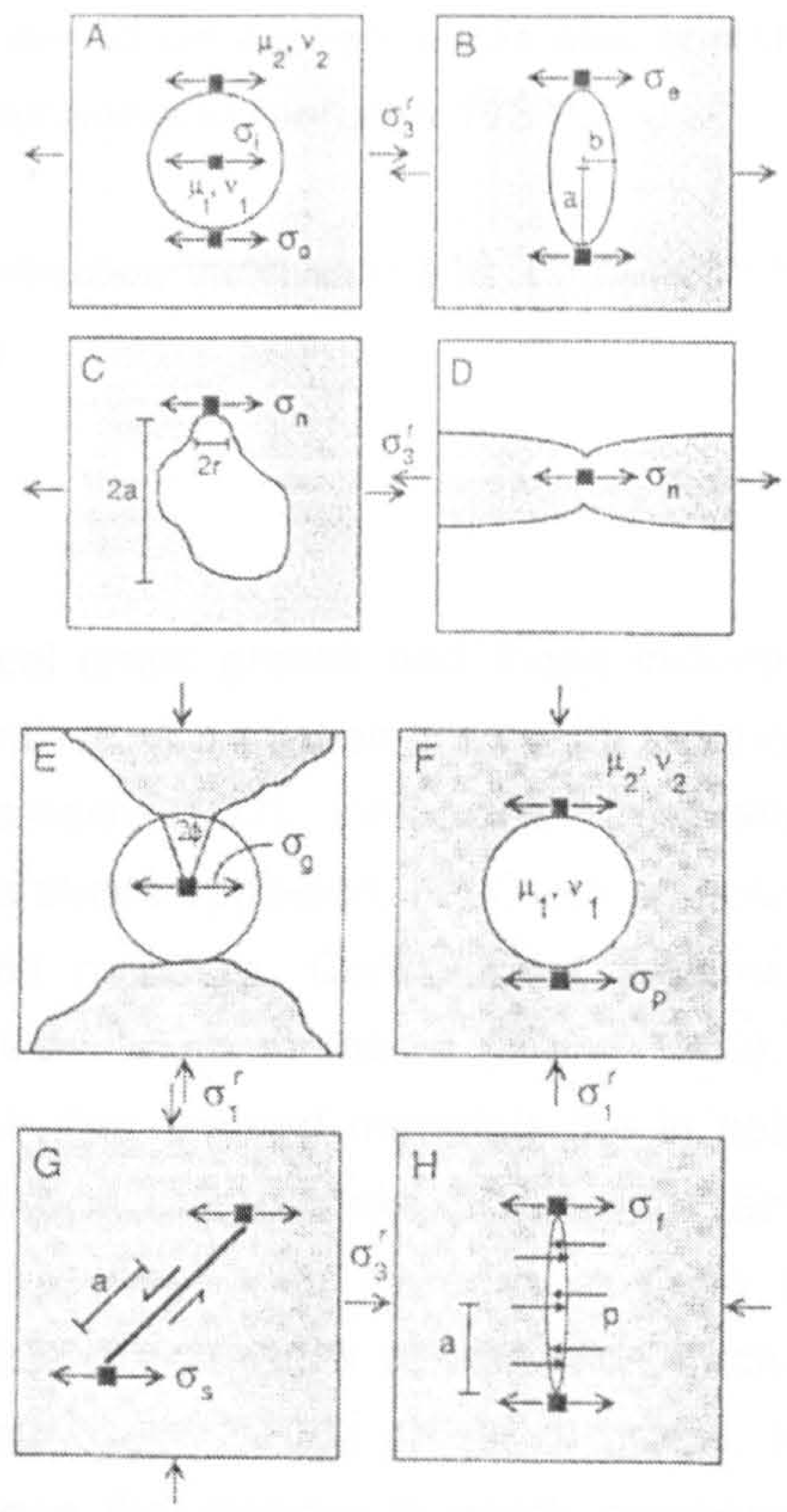
Whether crack growth is considered in terms of crack tip stress intensity or of extension force, crack growth has been observed at *subcritical* values. This is particularly true where long term loading has occurred producing time dependent cracking, and is enhanced where high temperatures and/or reactive chemical environments are involved. Lajtai (1977) envisages subcritical fracture development in a predominantly compressive stress field, aided by pore water pressure and time dependent stress corrosion, and Hoek (1968) also highlights the importance of rock moisture content. Several mechanisms for subcritical crack growth are recognised.

Figure 2.11 Idealisations of local stress intensity for different types of rock flaw.

A: Circular inclusion (eg fossil, grain, clast) under tension (σ_o);
B: Elliptical hole (eg cavities, microcracks, grain boundaries) under tension (σ_e);
C: Irregular cavity (eg notch) under tension (σ_n);
D: Cusp in a stretch layer (σ_n);
E: Circular grain compressed between two grains (σ_g);
F: Circular inclusion (eg pores, microcracks) under compression (σ_p);
G: Inclined elliptical hole under compression (σ_s);
H: Internally pressurised (eg salt or ice crystal growth, hydration pressure) elliptical hole perpendicular to a remote compression (σ_f).

For each case, a = length; μ = elastic shear modulus; ν = Poisson's ratio; σ^r = remote stress; p = internal fluid pressure

(after Pollard and Aydin 1988).



2.4.3.1 Mechanisms of subcritical crack growth

Stress corrosion: Strained Si-O bonds at crack tips react more readily to corrosive environmental agents than equivalent unstrained bonds. This reaction, which might be combined with other chemical reactions such as dissolution, produces a weakened state in rock and renders it more susceptible to fracture at lower stresses. Stress corrosion might occur under widely varying conditions and it is the corrosive which largely controls the rate of crack growth (Atkinson and Meredith 1987). Subcritical crack growth due to stress corrosion is probably responsible for some rock fracture normally attributed to frost shattering (Whalley et al 1982). For glass, a lower limit of K for which no propagation is observed has been identified and is known as the stress corrosion limit, K_0. It is not clear whether such limits exist in rock (Freiman 1984; Atkinson 1984).

Dissolution: There is evidence to suggest that in soluble materials, solution is accelerated at crack tips, leading to crack extension. Solubility of a material might be increased by an increase in the partial pressure of carbon dioxide, or by a decrease in temperature (Atkinson and Meredith 1987). Continued crack growth might be inhibited if the products of dissolution are not removed from the crack tip.

Diffusion: Materials can fail at high temperature due to mass transport with possible mechanisms such as grain boundary diffusion and vapour phase transport taking place. Cracks formed this way are characterised by irregularity of form due to unstable propagation, and have a large crack tip damage zone (Atkinson and Meredith 1987) This mechanism is unlikely to be important for near surface rocks.

Ion Exchange: Lattice strains might result from ion exchange, and this in turn can lead to crack growth. Modifications in the chemistry of crack tip fluids due to ion exchange can also contribute to hydrolysis, dissolution and stress corrosion effects (Atkinson and Meredith 1987).

Microplasticity: Crack growth can result from micro deformation such as might be derived from mineral twinning and kink band boundaries (Kranz 1983).

2.4.3.2 Factors affecting subcritical crack growth

There is a wide range of factors which affect subcritical crack growth and these include: (i) Stress intensity (for a given mode of failure). (ii) Temperature. A reduction in fracture toughness occurs with increasing temperature (Atkinson and Meredith 1987). Increasing temperatures usually enhance chemical activity and therefore enhance stress corrosion. (iii) Crack tip solution (affected by pH, solubility and reactions). (iv) Applied pressure. Confinement suppresses microcrack development and therefore increases the time taken to failure (Kranz 1983). (v) Grain size and shape. Isolated microcracks establish in fine grained materials but in coarse grained materials grain boundary cracks coalesce and form macrocracks (Rice and Freiman 1981). Others have argued that fracture toughness increases with increase in grain size because of increased tortuosity (Gesing and Bradt 1983). Propagation of extension fractures due to applied stresses at grain contacts is strongly influenced by the shape of grains, their packing and sorting. These characteristics also influence the manner in which microcracks coalesce and propagate into macrofractures (Gallagher et al 1974). (vi) Microstructure. Microstructure properties include pore structure and size distribution, density, fabric, mineral composition and variations, and total pore volume. The variability of these properties in rocks makes it extremely difficult to ascertain their precise role (eg Galos and Kertesz 1995). (vii) Residual stress. Residual strains in rock can cause a rock to be in a state of mechanical stress before any external pressure is applied. In this way, K can be reduced by 20% or more in rocks (Atkinson 1984).

2.4.4 The role of fractures in rock breakdown

Theoretical fracture mechanics considers the simplified model of isolated fractures in a single crystal at microscopic level, but the question is how does this relate to macrofracture development in response to rock weathering? Experimental work has shown that as a machined notch in rock is loaded, isolated microcracks develop around the crack tip in a region known as the *process or damage zone* (Hoagland et al 1973). As more and more microcracks are generated they begin to coalesce and form a macrocrack. This macrocrack continues to extend so long as $K_I = K_{Ic}$. It is thought that initial behaviour in the process zone is linear, but that non-linear behaviour follows, perhaps achieved by plasticity (Hoagland et al 1973; Atkinson 1987). Hoagland et al (1973) show that advancement of the crack tip and process zone will leave a relic zone of microcracking parallel to the macrofracture (Figure 2.12). This might lead to shallow surface spalling of fracture surfaces and might in part be responsible for fracture surface morphology.

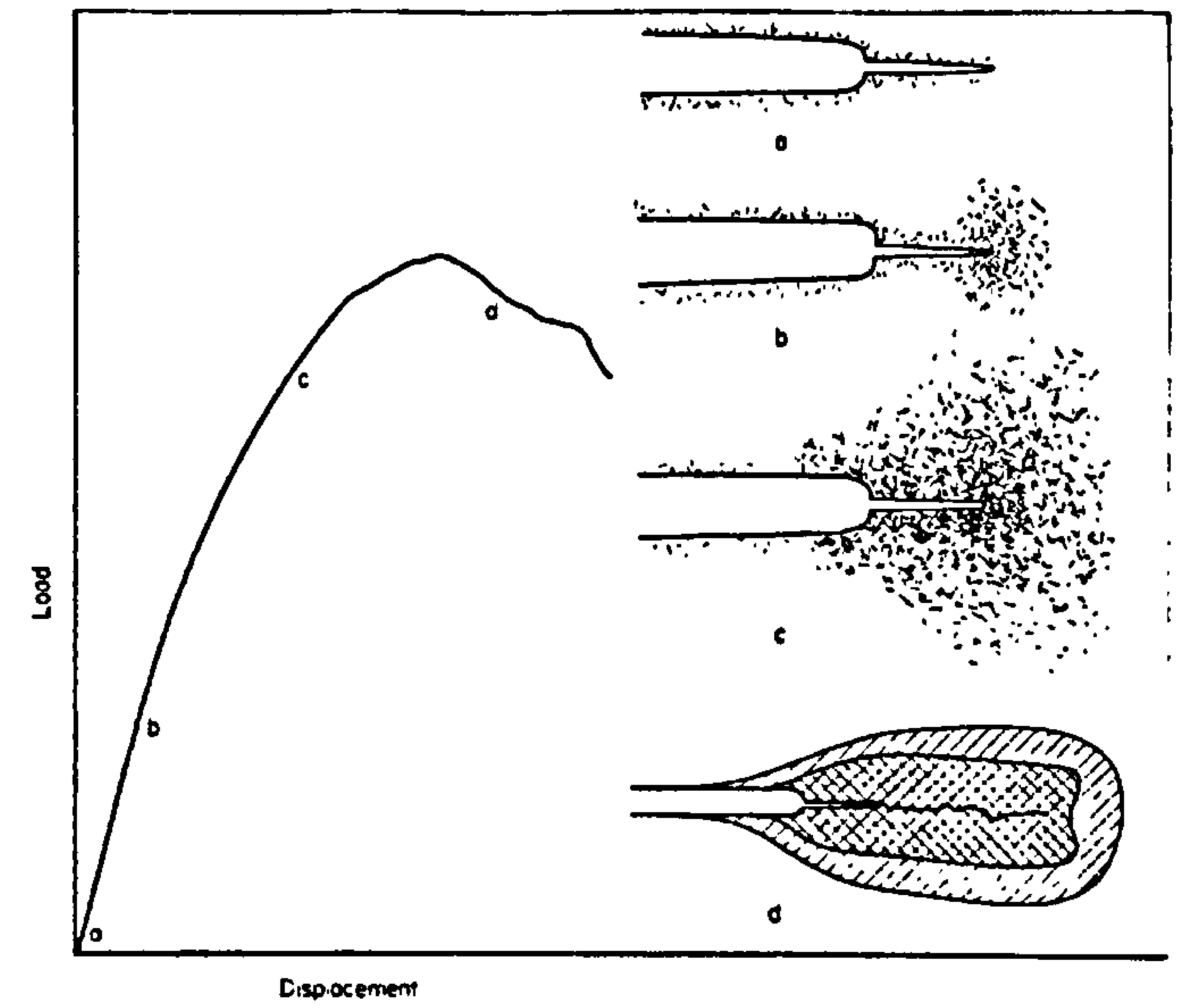


Figure 2.12 Damage zone development in an idealised rock fracture. Prior to application of stress (a), several random microcracks are present. Upon loading (b), new microcracks develop at the crack tip. This damage zone expands rapidly under increased loading (c) until inelastic behaviour occurs. Beyond peak stress (d), crack and damage zone extension occur simultaneously (*after Hoagland et al 1973*).

The propagation path of a fracture depends on the nature of stresses at the crack tip (Pollard and Aydin 1988), interaction with other cracks and flaws (Whalley et al 1982), and crack orientation relative to the direction of the applied stress field (Pollard and Aydin 1988). In a study of crack propagation in an oolitic limestone and a sandstone, Hoagland et al (1973) observed considerable crack growth in the intergranular cementing material, creating extremely tortuous paths. This was probably due to the relatively low strength of the cement compared with that of the grains. Intersection of macrofractures with the exposed rock surface might lead to detachment of fragments (eg Douglas 1981; Douglas et al 1991), and granular loss can occur by the isolation of grains due to grain boundary associated cracks, which might be single, isolated microcracks. The location, extent and depth of rock fragments detached by cracking is largely a function of the persistence and orientation of fractures. As already noted, these are largely related to the applied stress field, the chemical environment and the location and nature of pre-existing flaws.

With the exception of processes of chemical decomposition and minor forms of pore modification, it can be argued that most forms of rock weathering result in, and exploit, rock fractures at all scales. At the large scale, deep-seated fractures which relate to in situ gravitational and tectonic stresses can have a significant influence on landform evolution (eg Gerber and Scheidegger 1969, 1973). At the small scale, many fractures can be attributed to weathering processes. While physical processes dominate fracture development due to weathering, chemical processes are also important either in the direct alteration and decomposition of rock material, or in a stress corrosion role (Whalley et al 1982). The relationship between some weathering processes and fracture development is reviewed by Whalley et al (1982). They conclude that for the purposes of an investigation of rock weathering, three types of fractures are important: (i) pre-existing cracks which are extended by weathering processes and on which geomorphic studies have largely been focused; (ii) microcracks produced by deep-seated processes; (iii) concentrations of minerals known as potential 'weathering lines' which have the *potential* to become open cracks if acted upon by weathering processes. To a degree, all of these are addressed in this research.

2.4.5 Rock deterioration: modification of pores and microcracks

Conventionally, the effect of experimental weathering on rocks has been measured by macro properties such as weight change. In the last two decades a number of researchers have also used sonic velocity and elasticity as a means of measuring durability (eg Allison 1988, 1990; Remy et al 1994; Murphy and Inkpen 1996; Allison and Bristow 1999), and in this research, fracture density is additionally used. From the overview of literature presented above, it is clear that macrofractures and the detachment of rock fragments result from the initiation and propagation of cracks at the micro scale. These microcracks can develop from flaws including existing pores and cavities. Consequently, one might expect that if microcracks were being generated due to weathering this would be reflected in changes to void-dependent properties such as pore size distribution and total pore volume. This would probably also result in a reduction of rock strength and ought to be identifiable in measurements of sonic velocity. It is a pity that there not more examples in the published literature of experimental studies where rock properties such as these have been monitored throughout simulated weathering (eg Accardo et al 1981; Fitzner 1988). Several such measurements are made in this research (presented and discussed in Chapter Five) and the results used to make inferences about the internal mechanisms operating.

2.5 Implications For This Research

The core aim in the first part of this thesis is to investigate the influence of existing fractures, rock flaws and structure on the response to weathering at the material scale, and to describe the mode of rock breakdown which results. The role of other void-dependent (eg porosity, saturation coefficient, pore size distribution, microporosity), lithological (density and texture) and mechanical (elasticity, compressive strength, tensile strength and point load strength) properties is also examined since this can provide further insight into (i) the mechanisms involved in rock fracture due to weathering, and (ii) weathering mechanisms occurring under different environmental conditions. Four simulated weathering tests are used; freeze-thaw, salt weathering, wetting and drying and slaking. These tests broadly represent some of the key mechanical weathering processes believed to be active in the temperate climate of the UK, and the slake durability test additionally covers the erosive effects of impact and abrasion as might be expected at coastal locations. No attempt is made to simulate chemical weathering processes though their potential role is acknowledged, both in terms of material weathering and in terms of their contribution to stress corrosion. No attempt has been made to model precisely actual environmental regimes in designing the experimental weathering tests. Rather, the primary purpose has been to induce breakdown of the rocks.

CHAPTER THREE
SAMPLING, LABORATORY TECHNIQUES AND ROCK CHARACTERISATION

3.1 Introduction to the Experimental Programme

Ten sedimentary rocks were selected and subjected to experimental freeze-thaw, salt weathering, wetting and drying and slake durability. The experimental weathering tests are described in the next section and the measurements made, their timing and the total number of cycles conducted for each test are outlined in section 3.3. In section 3.4, three deterioration indicators, weight loss, fracture density and fracture porosity are described. The techniques used to measure rock properties are described in section 3.5 and the sampling and preparation of the rocks used is explained in section 3.6. In the final section brief lithological descriptions of the ten rocks are given.

Throughout Part One of the thesis the following codes are used to identify each of the rocks: Low density chalk - **LdCh**; magnesian limestone - **MagL**; oolitic limestone - **OolL**; high density chalk - **HdCh**; sparry limestone - **SpaL**; weathered sandstone - **WeaS**; calcareous sandstone - **CalS**; micaceous sandstone - **MicS**; laminated siltstone - **LamZ**; and metasediment - **MetS**. The descriptive terms adopted for the two chalks, low and high density, are relative. They relate to criteria set out by Mortimore and Fielding (1990), where the corresponding terms soft (LdCh) and hard chalk (HdCh) apply.

3.2 Experimental Weathering Tests

3.2.1 Freeze-thaw

3.2.1.1 Equipment, design and construction

A freeze-thaw chamber was constructed by modifying a standard domestic chest freezer as illustrated in Figure 3.1 and Plate 3.1. The freezing part of the cycle was achieved using a 24h electronic timer which permitted the compressor to be turned on and off at pre-set times at the maximum thermostat rating on 'Fast Freeze'. In addition to the compressor being switched off, the heating cycle was achieved by installing a 3kW immersion heater and pre-set thermostat in the chamber. The wattage of the heater was reduced by a 200VA 50V step-down transformer, to provide the amount and rate of heating required (around 130W). The heater was also turned on and off at pre-set times by a second electronic timer, so specimens were only handled when being removed for monitoring. The heating device was attached to an outer tray filled with a 2:1 mixture of water and standard ethylene glycol antifreeze. An inner tray was used to house the specimens which were placed on a plastic grid to prevent contact with the metal base, and immersed to 30mm in distilled water. Thus, during the freezing part of the cycle, the water immersing the specimens froze, and during the thawing part of the cycle, heat was conducted to the ice via the heated anti-freeze in the outer tray. Thus the thermal regime established in specimens was primarily via conduction rather than change in air temperature, which is a more realistic representation of natural conditions (Warke and Smith 1998). To ensure good

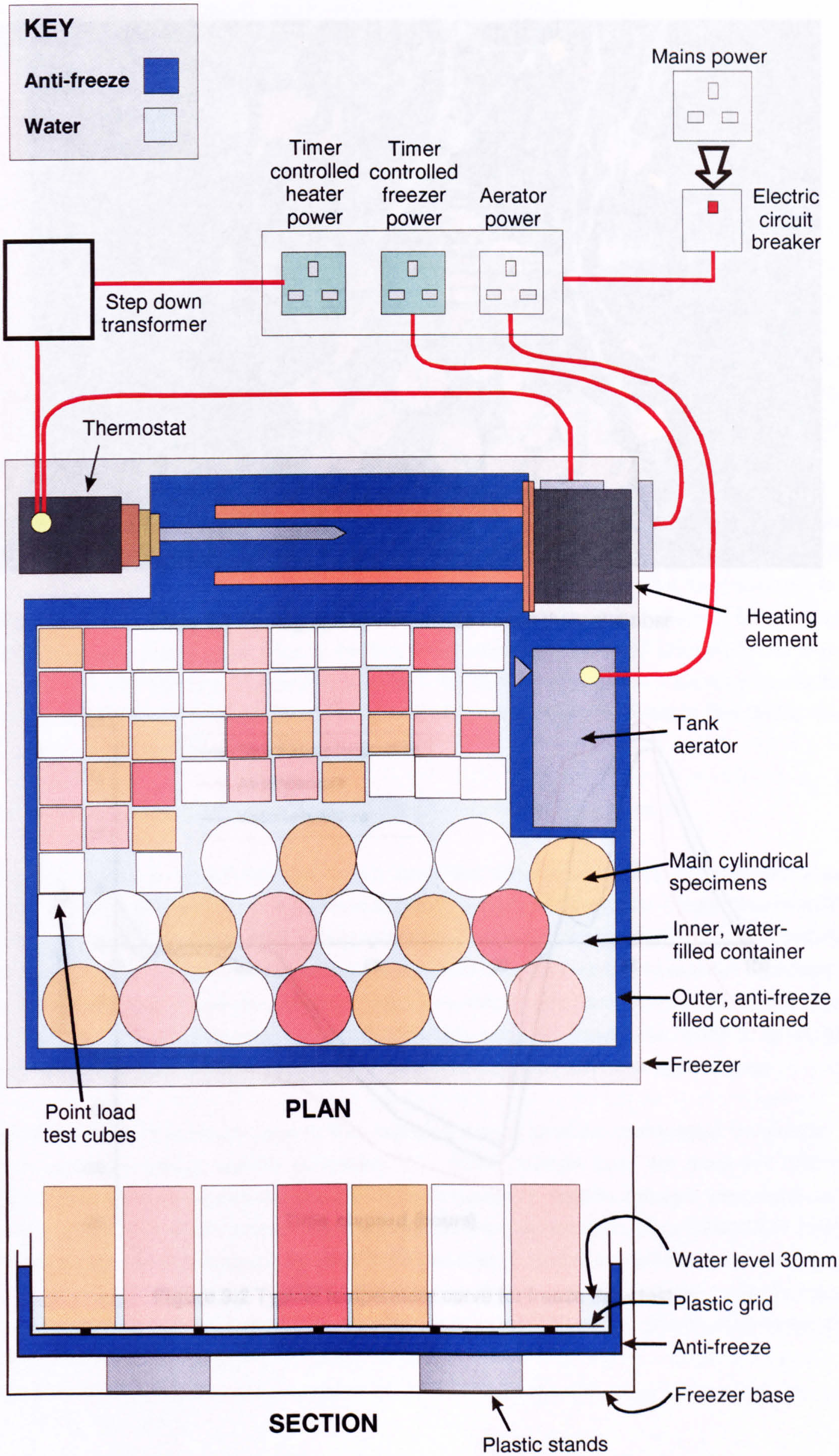


Figure 3.1 Experimental set up for freezing and thawing test

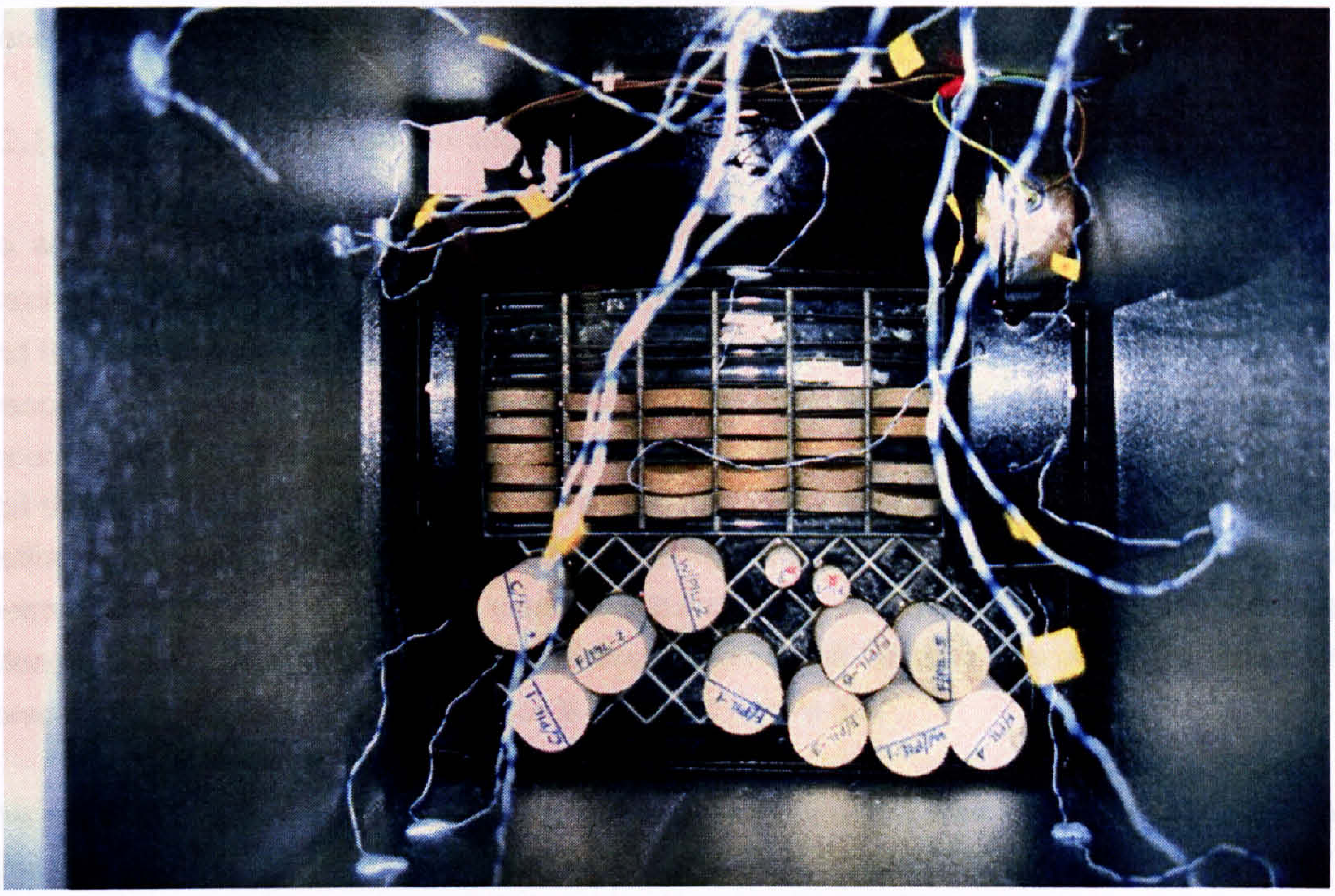


Plate 3.1 Photograph of view inside freeze thaw chamber

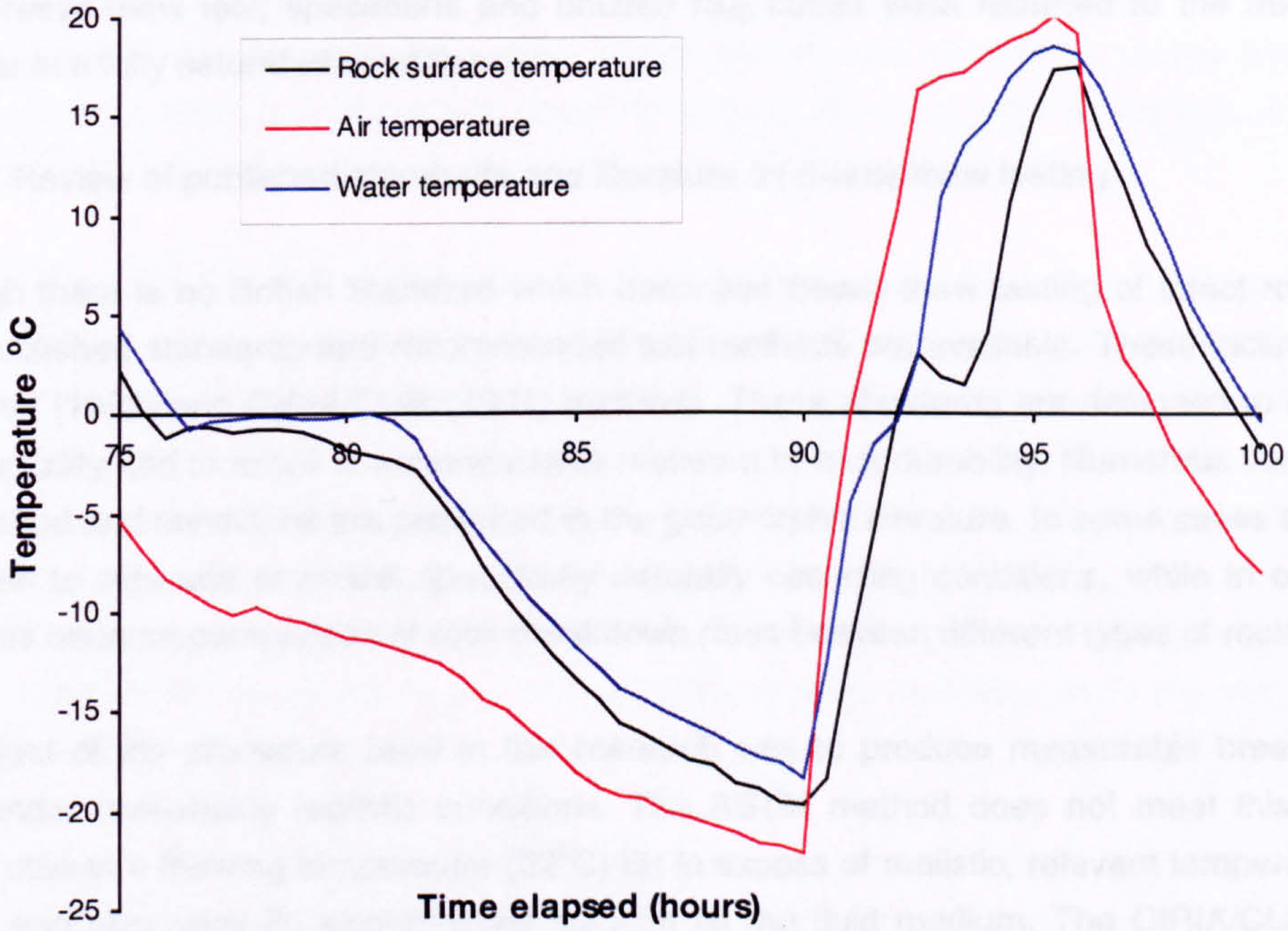


Figure 3.2 Typical temperature curve for freeze thaw test

circulation of the anti-freeze a small tank aerator was placed in the outer tray. The two trays were made of steel, welded at corners and edges and coated with Hammerite paint.

3.2.1.2 Experimental set-up, running and monitoring

To achieve the required rates of freezing (2°C/h) and thawing (6°C/h) and the necessary maximum (18±2°C) and minimum (-18±2°C) temperatures, freezing and thawing cycles of 18h and 6h, respectively, were established. This regime was selected to induce rock deterioration in reasonably realistic circumstances and was not an attempt to model specifically any naturally occurring regime. Temperatures of the anti-freeze, water, the surface of selected specimens and the chamber air were monitored in several locations at 30 minute intervals throughout all testing. This was achieved using 12 copper-constantan (an alloy of copper and nickel) thermocouples connected to an electronic datalogger. A separate temperature datalogger (Monolog) was used to provide a thermocouple reference temperature. A typical temperature versus time chart for one 24 hour cycle is shown in Figure 3.2.

The number of specimens representing each sample in this test was nominally five, but several exceptions exist and a detailed schedule of specimens is given in Table 3.1. Along with the main rock specimens, a selection of cubes for the measurement of point load strength was removed at intervals and oven-dried ready for testing. Where IS_{50} cubes were to be subject to further cycles of freeze-thaw prior to testing, they underwent identical saturation and drying procedures as for the main specimens in order to be fully representative. Following interruption to the freeze-thaw test, specimens and unused IS_{50} cubes were returned to the freeze thaw chamber in a fully saturated condition.

3.2.1.3 Review of published standards and literature on freeze-thaw testing

Although there is no British Standard which describes freeze-thaw testing of intact rock slabs, other published standards and recommended test methods are available. These include ASTM D5312-92 (1992) and CIRIA/CUR (1991) methods. These standards are designed to maximise reproducibility and to arrive at a comparative measure of rock durability. Numerous variations of test method and conditions are published in the geomorphic literature. In some cases the object has been to replicate or model specifically naturally occurring conditions, while in others the focus has been on comparison of rock breakdown rates between different types of rock.

The object of the procedure used in this research was to produce measurable breakdown in rocks under reasonably realistic conditions. The ASTM method does not meet this criterion since it utilises a thawing temperature (32°C) far in excess of realistic, relevant temperatures for the UK and also uses an alcohol-water solution as the fluid medium. The CIRIA/CUR (1991) method, based on draft methods for NEN 5184 and BS812, is also unrealistic since it involves a particularly rapid freezing rate (3°C/h), freezing in isolation from an external moisture supply (with a plastic film wrapped around the specimen preventing extrusion), and thawing by sudden immersion into warm water.

Sample Code	Number and type of large cylinders	
Low density chalk (LdCh)	5	
Magnesian limestone (MagL)	5	
Oolitic limestone (OolL)	5 + 1	Five specimens deteriorated severely after 11 and 21 cycles so an additional specimen was used to re-run the test over shorter interrupt intervals
High density chalk (HdCh)	9	
Sparry limestone (SpaL)	5	
Weathered sandstone (WeaS)	5	
Calcareous sandstone (CalS)	6	Specimen 1 had unusual weathering features (see pre-test characterisation in section 3.7.7 for further details).
Micaceous sandstone (MicS)	5	
Laminated siltstone (LamZ)	9	
Metasediment (MetS)	5	Due to difficulties in coring, rectangular saw-cut specimens were used.

Table 3.1 Schedule of test specimens for each sample

3.2.2 Salt weathering

The experimental method adopted for salt weathering was broadly based on ASTM designations C88-90 and D5240-92, which describe the method for the magnesium sulphate soundness test. Magnesium sulphate was selected for use in preference to sodium sulphate because it produces a more severe reaction (Goudie et al 1970; Goudie 1974; ASTM D5240 1992). The magnesium sulphate solution was prepared from the addition of anhydrous sulphate ($MgSO_4.7H_20$) to distilled water at 25-30°C in the ratio 1400g/litre water. The mixture was regularly stirred to encourage dissolution of salt crystals and allowed to cool to 21±1°C. The solution was covered and allowed to stand for at least 48 hours prior to use and any surface cake which developed was broken up.

For each sample five oven-dried specimens (with the exception of LamZ where four specimens were used) were placed in individual mesh bags and immersed in the solution for a period of 16 to 18 hours (Figure 3.3). Following this period, specimens were removed from the solution, permitted to drain for 15 minutes, taken from their mesh bags and placed on trays in the drying oven, pre-heated to 105±2°C. Specimens were dried until constant weight was achieved. To repeat the cycle, oven-dried specimens were cooled at room temperature before replacing in their mesh bags and re-immersing in the solution. The procedure was repeated for a total of five cycles. After each interruption to the test, specimens were re-immersed in the solution in an oven-dried condition.

At interruptions and after the final cycle, specimens were washed until the complete removal of sulphate solution was achieved as determined by the reaction of wash water to a solution of barium chloride ($BaCl_2$). Specimens were washed initially in distilled water and later in tap water at 43±6°C. The hot water was introduced at the bottom of a tank containing the specimens and allowed to overflow at the top. Porosity and weight loss data suggest that the complete removal of salts was not successful for all the samples tested. The magnesium sulphate soundness of specimens was determined by re-weighing the dried sample after final oven-drying and was determined as follows:

$$\% \text{ Soundness} = 100 - \left(\frac{A-B}{A} * 100 \right) \qquad [3.1]$$

Where A is the pre-test oven dried mass, and B is the post-test oven dried mass.

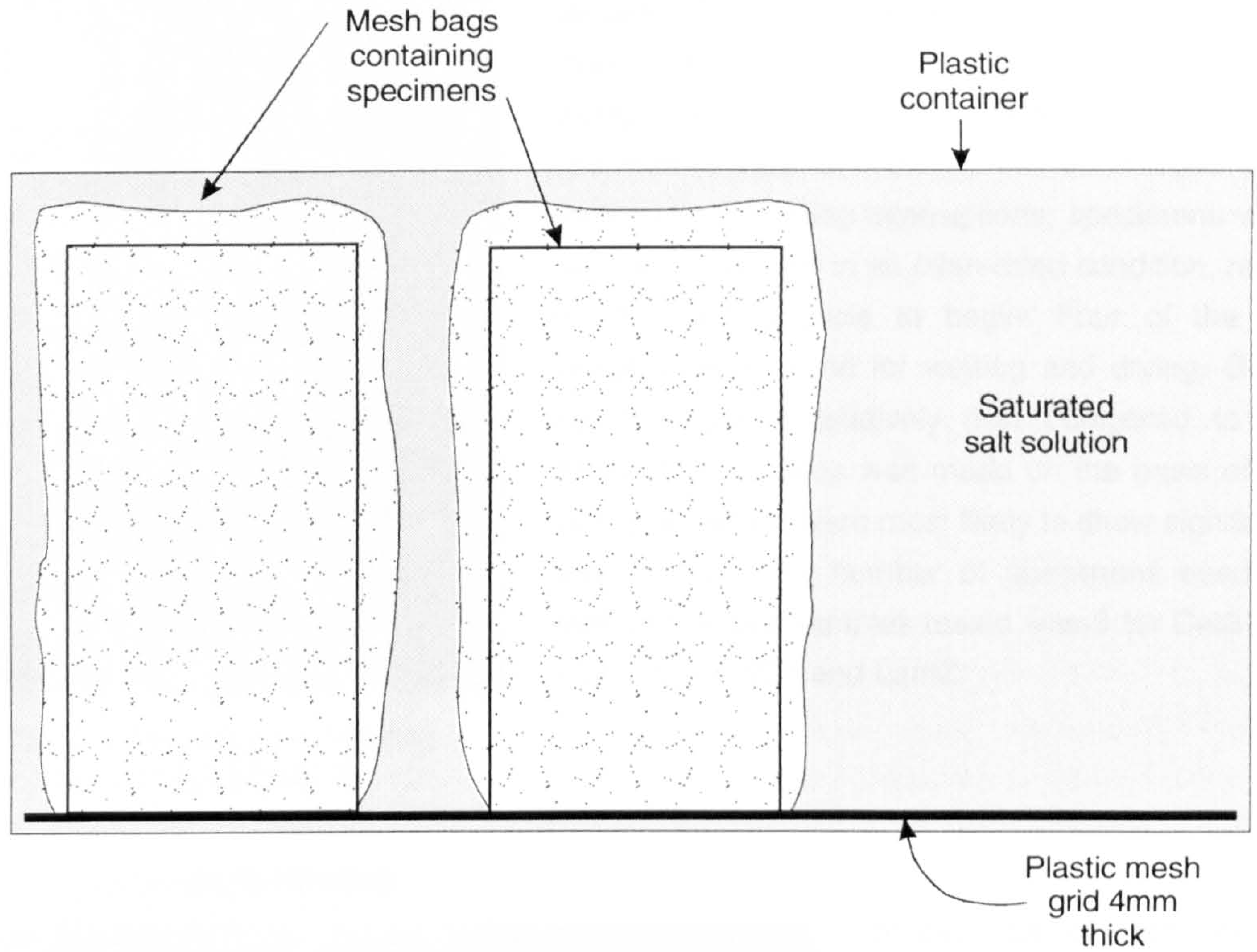


Figure 3.3 Diagram of specimen immersion in saturated solution for salt weathering test

3.2.2.1 Review of published literature

In some respects the method used differs considerably from methods described in the rock weathering literature. In particular, specimens were immersed in the solution for much longer than reported elsewhere (eg Goudie 1974; Cooke 1979; McGreevy 1982; Smith and McGreevy 1983), although Goudie (1999) uses a similar immersion period. Furthermore, the availability of climatic cabinets in other investigations has meant that variable temperature and humidity regimes were possible during the drying part of the cycle. Cooke (1979), for example, utilised a cyclic temperature range from 20 to 70°C and relative humidity from 20 to 85% to simulate conditions for a particular Egyptian wadi under investigation. A similar regime was adopted by Goudie et al (1970), Goudie (1974) and Smith and McGreevy (1983). The procedure adopted here ensured that all of the rocks tested would undergo some deterioration over the finite number of cycles used. There was no attempt in designing temperature and saturation duration regimes to simulate real conditions.

3.2.3 Wetting and drying

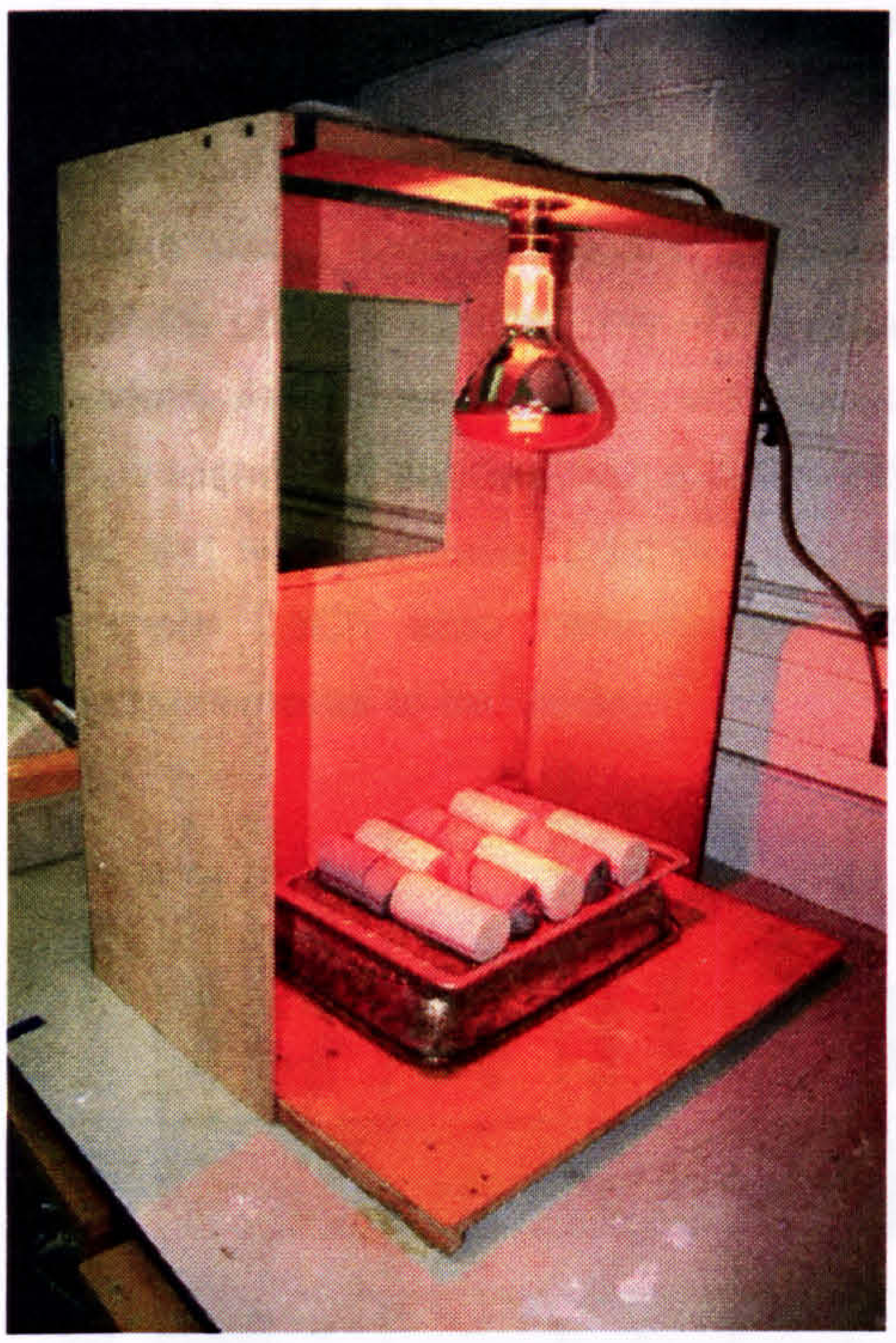


Plate 3.2 Photograph of wetting and drying apparatus

The wetting and drying test was set up largely in compliance with ASTM designation D5313-92. Specimens were placed on a 6mm layer of coarse sand and dried 45cm beneath a single 150w infra-red lamp (Plate 3.2). Following drying for at least 6h (maximum 10h), specimens were removed from the drying tray and immersed in water for at least 12h. Specimens were sometimes left submerged over weekends. Following interruptions, specimens were returned to the test in an oven-dried condition, ready for the wetting cycle to begin. Four of the ten samples were tested for wetting and drying. Given that the test is relatively mild compared to the others, the selection was made on the basis of the rock types which were most likely to show significant deterioration. The number of specimens used for each of the four samples tested was 3 for CalS and 5 for LdCh, HdCh and LamZ.

3.2.4 Slake durability testing

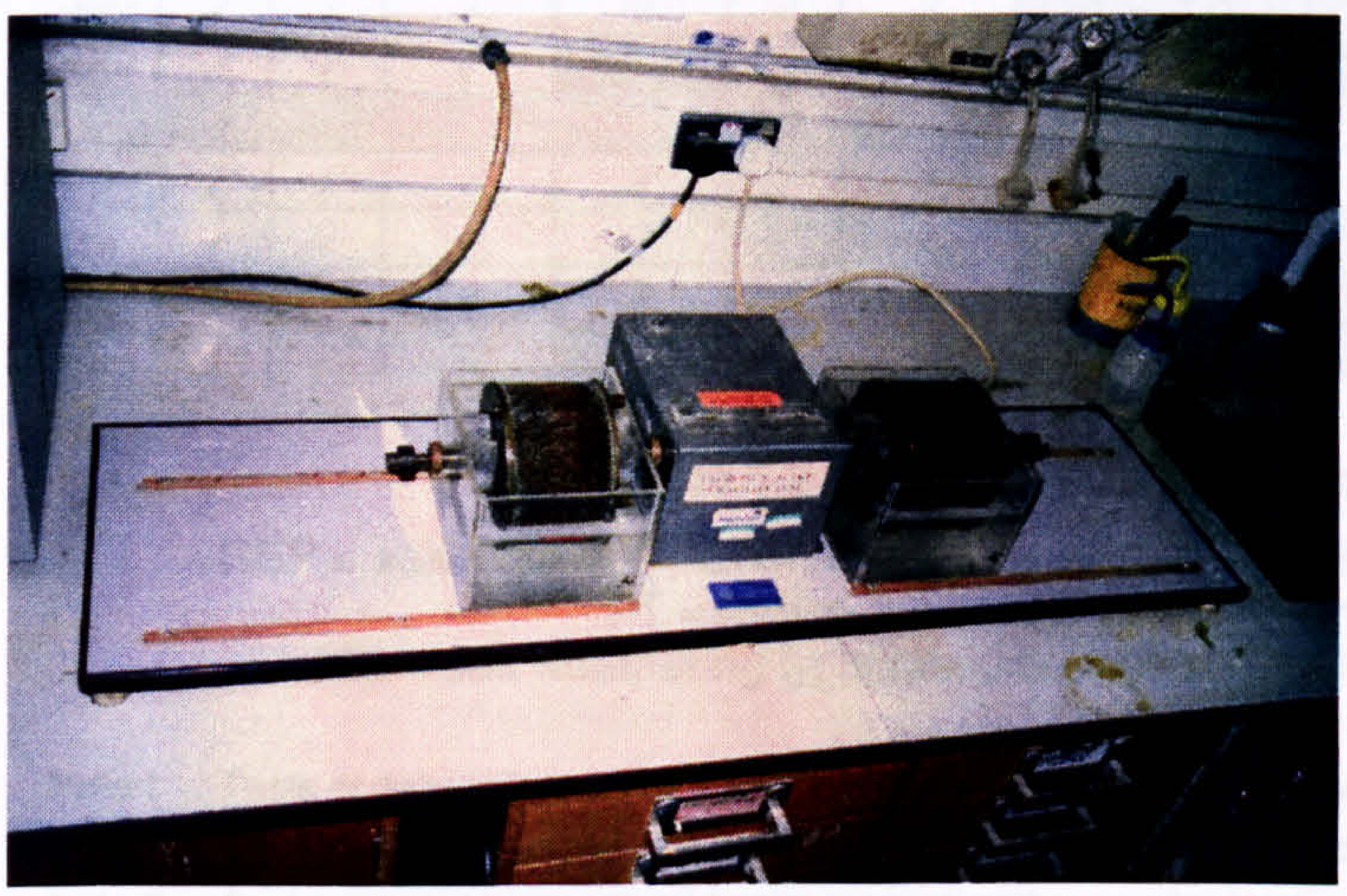


Plate 3.3 Photograph of the slake durability test apparatus

For each sample, ten saw-cut cubes of nominal dimension 30mm were subjected to a standard slake durability test following the recommendations of Franklin and Chandra (1972). The test provides an index of rock susceptibility to slaking by subjecting samples to two cycles of rotation in a drum partially immersed in water (Plate 3.3). For each sample, ten specimens of similar dry mass are used, giving a total mass of 450-550g. Slake durability is not only a reflection of resistance to wetting and drying (slaking), but also to abrasion and impact. For this study, five cycles of slaking are used instead of the standard two, as recommended by the Brown (1981) for higher durability rocks. Following interruptions to the test, specimens were returned in an oven-dried condition.

3.3 Deterioration Monitoring Schedule

Table 3.2 indicates what rock properties were measured, at what stage in the experimental procedures, and in relation to which of the four weathering tests. The total number of freeze thaw cycles to which each sample was subjected (given at the end of each histogram bar in

Figure 3.4) was determined by the point at which (a) sample deterioration was so severe as to render it difficult to extract further meaningful data, or (b) further, significant deterioration was considered unlikely to occur over a reasonable timescale. The timing of test interruptions (refer again to Figure 3.4) was based on a pre-test estimate of the behaviour of each rock type under experimental weathering and also from regular inspections of specimens during testing to monitor change qualitatively. This approach might appear somewhat trial and error but mostly worked well. One exception was sample OolL which deteriorated very severely in the freeze-thaw test much sooner than anticipated so a single extra specimen (OolL(2)) was used to re-run the test with a smaller total number of cycles and with shorter periods between interruptions. Sample LdCh also deteriorated too severely under freeze-thaw for some monitoring measurements to be made.

Monitoring	Weight Loss[1]	Dry Density[1]	Effective Porosity[1]	Fracture Density[1]	Vp and Vs[1]	Macro Description[1]	Compressive Strength[2]	Tensile strength[2,3]	Point load strength[2,3]	Mercury Porosimetry[4]	SEM[4]	Petrographic Analysis[4]	Photograph[1]
Pre-Weathering													
REP							✓	✓	✓	✓	✓	✓	
FT		✓	✓	✓	✓	✓							✓
MS		✓	✓	✓	✓	✓							✓
WD[5]		✓	✓	✓	✓	✓							✓
SD		✓	✓			✓							✓
During Weathering													
FT	✓	✓	✓	✓	✓	✓		✓[6]	✓				✓
MS	✓	✓	✓	✓	✓	✓							✓
WD[5]	✓	✓	✓	✓	✓	✓							✓
SD	✓	✓	✓			✓							✓
Post-Weathering													
FT	✓	✓	✓	✓	✓	✓		✓[6]	✓	✓[7]	✓	✓	✓
MS	✓	✓	✓	✓	✓	✓				✓	✓	✓	✓
WD[5]	✓	✓	✓	✓	✓	✓							✓
SD	✓	✓	✓			✓							✓

Table 3.2 Schedule of specimen monitoring

REP = values or descriptions *representative of the whole sample* were obtained.
FT = freeze-thaw; WD = wetting and drying;
MS = salt weathering (magnesium sulphate); SD = slake durability test.

Note 1 Tests or descriptions, as appropriate, were undertaken for each individual test specimen in each rock.

Note 2 Extra test cylinders, discs and cubes were cut from the main sample for these destructive tests.

Note 3 Post-test tensile strength and point load strength were obtained only for freeze-thaw.

Note 4 These tests were undertaken on a 'before and after' basis only for the freeze-thaw and salt weathering tests. Pre-weathering test pieces were taken from the main sample and post-weathering test pieces were cut from representative, weathered specimens.

Note 5 Only four of the ten samples (LdCh, HdCh, CalS, LamZ) were tested for wetting and drying.

Note 6 Samples HdCh and LamZ only.

Note 7 All samples except SpaL and MetS.

For the salt weathering test, five cycles were used for all samples. The test was interrupted after one cycle and again after three cycles, with the exception of LamZ and HdCh which were only interrupted once, after two cycles.

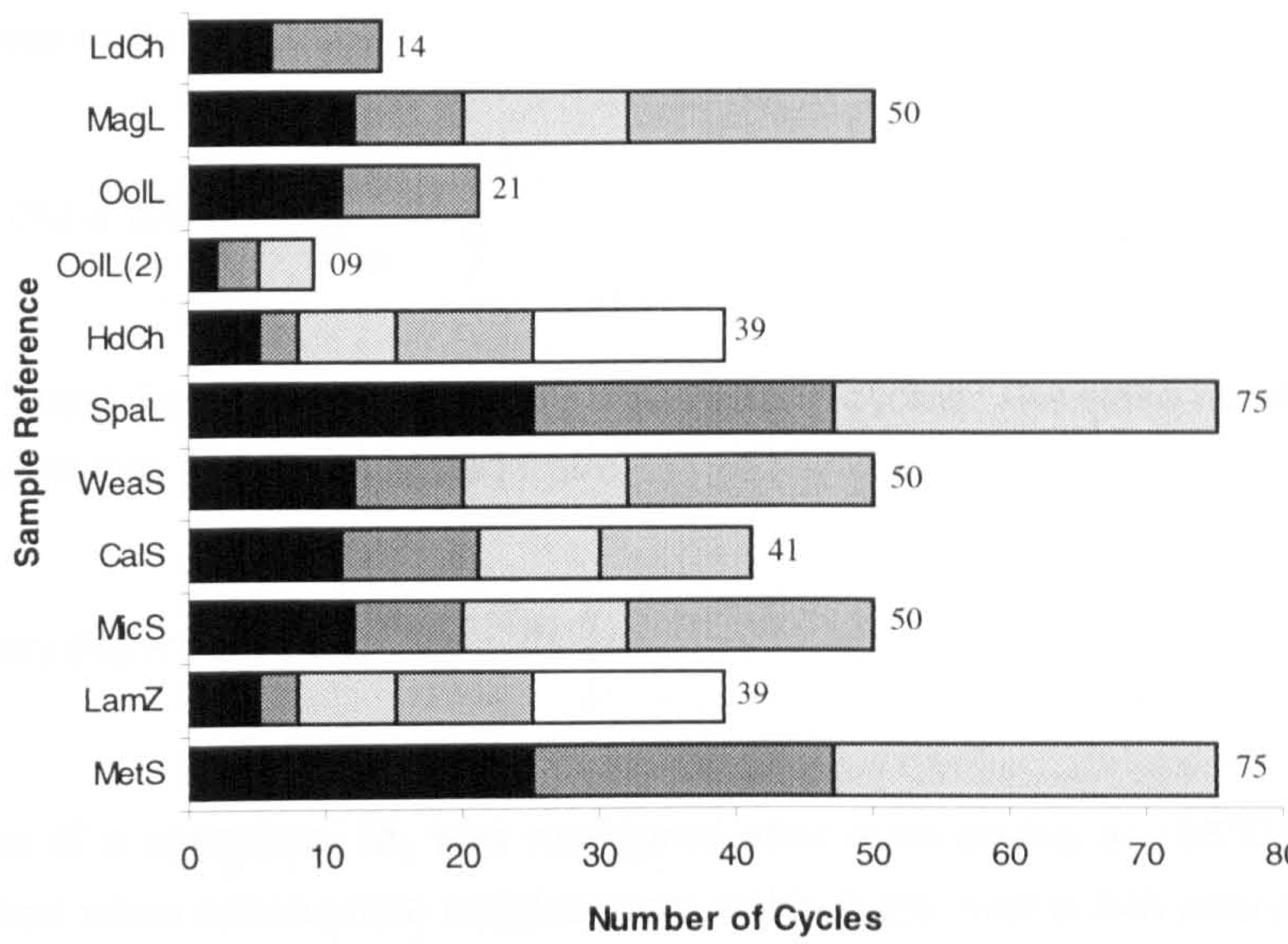


Figure 3.4 Total number of cycles conducted and timing of test interruptions for freeze-thaw.

For the wetting and drying test, the total number of cycles used is shown in Figure 3.5, together with the timing of interruptions for monitoring. Both were based on the same considerations as for the freeze thaw test.

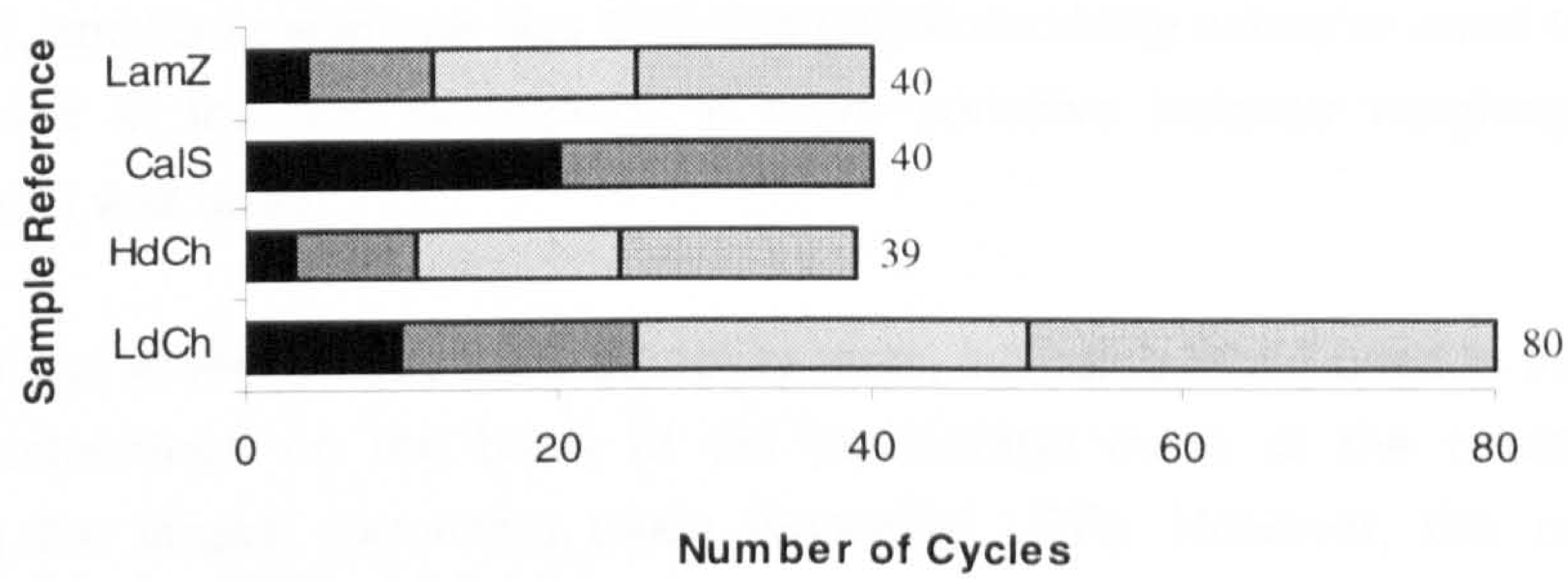


Figure 3.5 Total number of cycles conducted and timing of test interruptions for wetting and drying.

Five cycles were used for slake durability testing, with one interruption after two cycles.

3.4 Indicators of Deterioration

Determination of the severity of deterioration is strongly influenced by the properties actually measured. Conventionally, percentage weight loss has been used as an indicator of deterioration. However, there are limitations in using weight loss as the sole measure of rock durability. It quantifies the extent of material *detachment*, but does not reflect (i) in situ breakdown by fracturing and weakening, or (ii) deterioration at the microscale. For this reason, two further indices of deterioration are used in this research as explained below.

3.4.1 Weight loss

Weight loss was calculated from:

$$\text{Weight Loss (\%)} = 100 * \left(\frac{M_{d0} - M_{dn}}{M_{d0}} \right) \qquad [3.2]$$

Where M_{d0} = initial dry mass (g), M_{dn} = dry mass after *n* cycles. The slake durability index is the inverse of weight loss and is calculated from:

$$\text{Slake Durability (\%)} = 100 * \left(1 - \frac{M_{d0} - M_{dn}}{M_{d0}} \right) \qquad [3.3]$$

The dry mass of a specimen, M_d was measured after oven drying at 105°C±2°C to constant weight (reached when consecutive weights were within 0.2% over a 24h interval). It is possible that subjecting rock specimens to indirect heating at this temperature could induce damage in the form of thermal cracking. However, a range of rocks exposed to high temperatures in studies of the effect of fire on rock weathering showed negligible change in rock properties at 100°C (Goudie et al 1992; Allison and Goudie 1994; Allison and Bristow 1999). Case hardening was offered as an explanation for one exceptional specimen which showed an *improvement* in Young's dynamic modulus at 50°C.

For specimens exceeding 300g dry mass, a heavy-duty balance weighing to ±0.5g was used (±0.1% accuracy), and for specimens less than this (slake durability cubes or small fragments of specimens retained at the end of testing), a more sensitive balance weighing to 0.001g (±0.003% accuracy) was used.

For many British and American standard durability tests, including those used in this research, weight loss is determined on the basis of the percentage mass of the retained portion, calculated using the largest remaining piece (hereafter LRP). However, this can be very misleading, particularly when a specimen breaks into a very few pieces of similar size. The weight loss, and consequently susceptibility to weathering, is then grossly over-estimated. A modified weight loss criteria is therefore used in this research, where all fragments exceeding 10% of the initial specimen dry mass contribute towards the retained portion. Similar, though not identical criteria, have been adopted in other rock weathering studies (eg Goudie 1974, 1999; Williams and Robinson 1998). For comparison only, LRP weight loss values are also provided.

3.4.2 Fracture density (F_D)

Fracture Density measures the extent of in situ fracturing visible on specimen surfaces. Specifically, it represents the total surface area of fractures (mm^2) per unit volume of rock (mm^3) and can be obtained by simple macro-analysis. The method is based on stereological principles, enabling interpretation of three-dimensional objects, in this case fractures, on the basis of two-dimensional observations. Fractures are assumed to be obloid in three-dimensional form (ie penny shaped in plan and elliptical in cross section).

The surface area to volume ratio (Sv) for planes in a given volume of material can be determined from the relationship $2P_L$, where P_L is the number of point intersections (P) per unit length ($_L$) of grid line (Underwood 1970). This equation was originally derived by Saltykov in 1945 and has been re-derived on a number of occasions since (see Underwood 1970 for a full review). The equation is valid for isolated and networked continuous and discontinuous surfaces. It is normally used to evaluate the density of surfaces in space by counting point intersections on grid lines drawn on section planes through the medium.

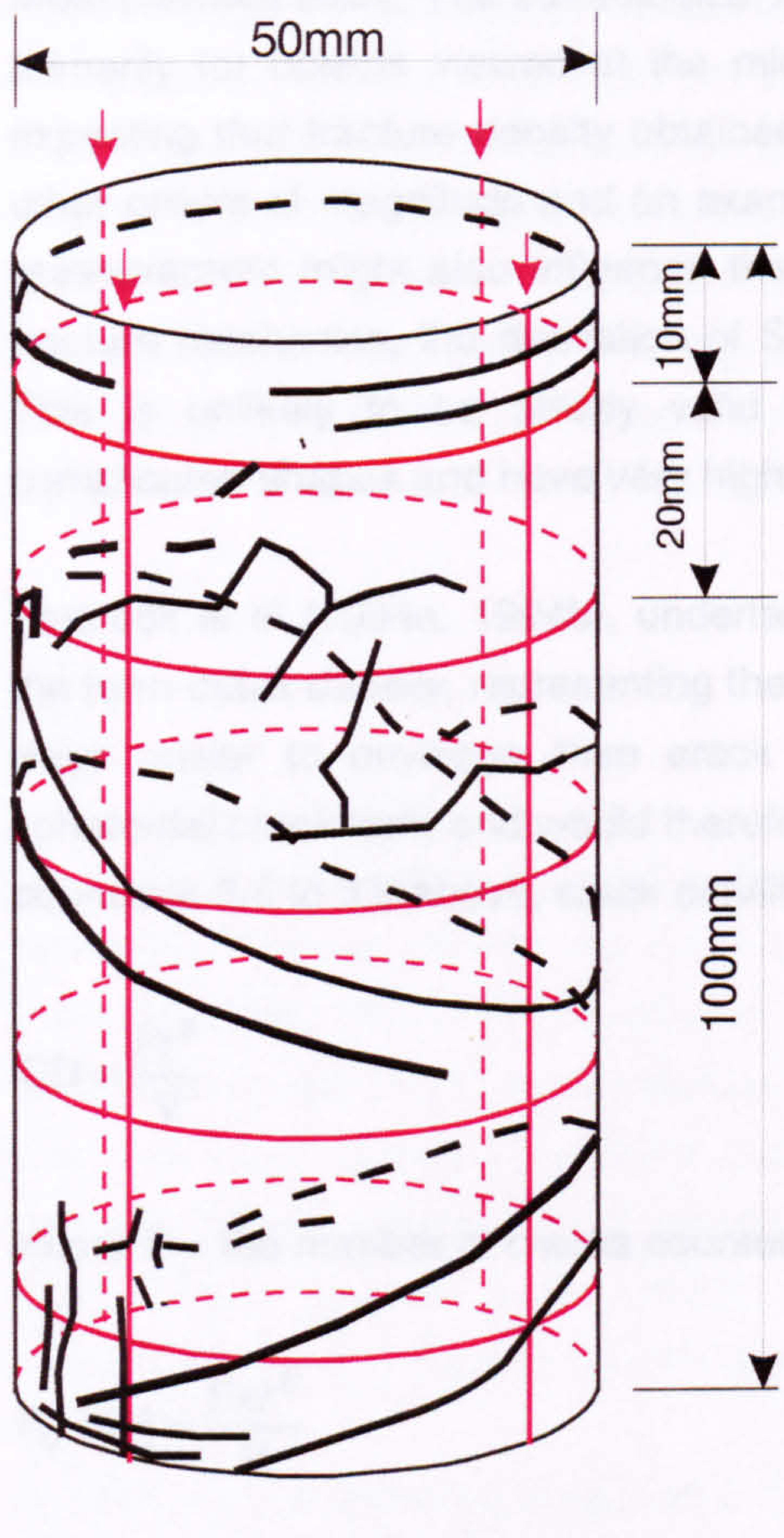


Figure 3.6 Grid superimposed on rock specimens for point counting of intersections

In this case, the method of point counting has been modified (i) to suit the cylindrical form of the rock specimens used and (ii) to enable non-destructive monitoring throughout experimental weathering. A standard grid was superimposed on the surface of each specimen as indicated by the red lines in Figure 3.6: Four equi-distant, axial grid lines were established around the circumference of the specimen and five diametral grid lines were established along its length. The latter were spaced such that the top and bottom lines were 10mm from the edge of the specimen, with intermediate lines equally spaced between. For a small number of sub-length specimens (around 90mm), four diametral lines only were used to give a nominal spacing of 20mm. A count was made of the number of point intersections (P) of all fractures visible to the naked eye, and the total length of grid ($_L$) measured with a calliper to an accuracy of 0.01mm, enabling Sv to be determined from $2P_L$. This quantity (Sv) is hereafter referred to as Fracture Density (F_D).

Since one of the assumptions in the relationship $F_D = 2P_L$ is that fractures are circular in plan, it is possible to determine an index of mean fracture length (F_L). If F_D represents the *total* surface area of fractures per unit volume of rock ($_V$), then:

$$F_D = \frac{F\pi r^2}{V} \qquad [3.4]$$

(after Peacock et al 1994a; 1994b) where F = the number of fractures counted, and πr^2 = the surface area of a single fracture plane, and:

$$\text{Mean fracture surface area } \overline{S} = \frac{(F_D * V)}{F} \qquad [3.5]$$

An index of fracture length (F_L), representing the *diameter* of circular fractures, is found from:

$$2*\left(\sqrt{\frac{\overline{S}}{\pi}}\right) \qquad [3.6]$$

3.4.2.1 Potential sources of error

Measurement scale: The stereological basis upon which the technique has been derived works primarily for objects viewed at the microscopic scale. However, there are good reasons for expecting that fracture density obtained at one scale will be comparable with that obtained at other orders of magnitude and an example of this is discussed in section 4.2.7.2. The scale of measurement might also influence the form of fractures. In line with common convention in fracture mechanics, the derivation of Sv assumes an oblate, spheroidal form for microcracks. This is unlikely to be strictly valid for macroscale fractures, which sometimes assume complicated shapes and have very high aspect ratios.

Peacock et al (1994a, 1994b), undertaking a microscopic study of Carrara marble, developed the term *crack density*, representing the crack volume to sample volume ratio. Although in some ways easier to envisage than crack area to volume ratio, this concept assumes a true spheroidal crack form and would therefore not be appropriate to use here. Using the notation of equations 3.4 to 3.6 above, crack density is:

$$CD = \frac{Fr^3}{V} \qquad [3.7]$$

where F = the number of cracks counted and r = crack radius, and since

$$F_D\ (Sv) = \frac{F\pi r^2}{V} \qquad [3.8]$$

then

$$CD = \frac{rF_D}{\pi} \qquad [3.9]$$

Fracture length: The size of grid squares used is such that some small cracks avoid intersection and thus are 'missed' in point counting. This is counter-acted in part by the fact that small fractures which do intersect with the grid tend to lead to over-estimation of F_D. Large fractures, in contrast, tend to under-estimate F_D.

Stereological basis: Stereological analysis is designed for observation of two-dimensional objects in a three-dimensional media using single and serial sections through that media, or by utilising multiple visual angles or projections of it (Underwood 1970). In this case, sectioning of specimens was inappropriate for a non-destructive monitoring technique. In effect, the grid used represents serial projections on the surface of a three-dimensional medium incorporating axial and diametral (orthogonal) transects.

3.4.3 Ultrasonic pulse propagation velocity

3.4.3.1 Velocity indices

Because sonic wave propagation velocity in rock is dependent on a wide range of rock properties (see section 2.2.2), its use as an indicator of durability and soundness has long been recognised. The measurement of the velocity of sonic wave propagation through rock is a non-destructive technique which has the potential to detect deterioration of rock properties even where there is no visible evidence available (eg Goudie 1999). This contrasts with weight loss, which considers detached material and fracture density which considers in situ fracturing visible in hand specimen. Onodera (1963) proposed a Fracture Index, defined as the ratio of P-wave velocity in the rock mass to P-wave velocity in intact material. Deere at al (1967) later showed that:

$$\left(\frac{V_f}{V_l}\right)^2 = RQD \qquad [3.10]$$

where V_f is in situ (ie measured in the field) P-wave velocity, V_l is intact (ie measured in the laboratory) P-wave velocity and RQD is Rock Quality Designation. This relation is known as the Velocity Index. Work by Sjϕgren et al (1979) on igneous and metamorphic rocks also established a good correlation between P-wave velocity and fracturing, but in some cases found that rock mass quality was better expressed by using both compressional and shear wave velocity. Other authors (eg Cratchley et al 1972) have been unable to find convincing relationships between sonic velocity and degree of fracturing and it would appear that correlation between the two is more complex than first thought (Crampin 1981).

In 1976, Fourmaintreaux proposed a Quality Index (IQ), defined as the ratio of the theoretical velocity of the unaltered rock to actual velocity:

$$IQ = \frac{Vp}{Vp_o} * 100 \qquad [3.11]$$

where Vp is the *measured* P-wave velocity, and Vp_o is the *calculated* P-wave velocity based on the theoretical velocities of individual mineral constituents. Thus, IQ indicates in situ alteration of mineral constituents and cementing material from their unaltered state, together with rock porosity. A Deterioration Index (ID) is presented here, defined as the ratio of the P-wave velocity measured prior to durability testing (Vp_{init}) to P-wave velocity measured after n cycles of testing (VP_n):

$$ID = \left(1 - \frac{Vp_n}{Vp_{init}}\right) * 100 \qquad [3.12]$$

Thus ID is a reflection of the amount of rock deterioration induced by experimental weathering or durability testing. If used to compare deterioration in rocks with similar values for Vp_{init}, this index provides a simple and useful tool for comparative analysis. However, in rocks with widely dissimilar Vp_{init} values reduction in wave propagation velocity due to deterioration is not directly comparable in absolute terms. This is because, when a fracture is introduced into a rock which is relatively dense and has a high Quality Index, the absolute reduction in velocity is much greater than it would be if an identical fracture were introduced into a much less dense rock with a low Quality Index. This reflects the fact that the amount of change in velocity due to deterioration is a function of the pre-weathering state of the rock (ie in terms of fracture state and porosity). The net effect of this is that, although the Deterioration Index can be usefully used to compare specimens from the same sample, it should not be used to compare rocks with widely dissimilar pre-weathering velocity values.

3.4.3.2 Fracture porosity

In an attempt to overcome this inadequacy, an Index of Fracture Porosity (IF_p) is proposed. McDowell (1993) presents a time average formula [equation 3.13] to illustrate the way in which fractures influence wave propagation through a rock mass:

$$\frac{L}{Vp_1} = \frac{nw}{Vp_2} + \frac{(L - nw)}{Vp_3} \qquad [3.13]$$

where L = length of direct wave path (m); n = number of fractures; w = mean width of fractures (m); Vp_1 = P-wave velocity of the bulk rock mass (ie rock material *and* fracture infill); Vp_2 = P-wave velocity of joint infill only; and Vp_3 = P-wave velocity of intact rock material only. For instance, an intact rock mass has a measured velocity (Vp_3) of $4000ms^{-1}$. Into this rock mass, 10 air-filled fractures are introduced (Vp_2 = $330ms^{-1}$), each with an aperture of 0.05m (w). The length of the direct wave path is 20m. Using the equation above, the predicted velocity of the rock mass including fractures (Vp_1) will be $3130ms^{-1}$. If the fractures had been filled with water (Vp_2 = $1450ms^{-1}$), the resulting Vp_1 value would have been $3840ms^{-1}$, indicating that there is a considerably greater attenuation for air-filled fractures than for water-filled fractures.

If the terms in equation 3.13 are modified such that $Vp_1 = Vp_n$ (pulse velocity after 'n' cycles of weathering); $Vp_2 = Vp_{air}$ (velocity of air infill); and $Vp_3 = Vp_{init}$ (initial, or pre-test velocity), then the equation can be re-written to determine the total volume of new fracture porosity introduced as a result of experimental weathering. Since the number of fractures introduced to the rock specimen is unknown in this context, the value for n is taken to be one and can, therefore, be disregarded, thus nw becomes w:

$$w = \frac{L\,Vp_{air}(Vp_{init} - Vp_n)}{Vp_n(Vp_{init} - Vp_{air})} \qquad [3.14]$$

The term w then, represents the width of one, or several proportionately smaller, parallel, hypothetical fractures lying perpendicular to the direction of wave travel (across the diameter of the specimen). Since Vp_{init} reflects the inherent porosity and fracture state of the rock prior to experimental weathering, w *only* reflects *new void* introduced to the specimen as a direct result of experimental weathering. If it is also assumed that any hypothetical fracture is planar and that it fully transects a cylindrical core specimen, it is possible to calculate an absolute fracture volume (cm^3), though ideally, the Index of Fracture Porosity (IF_p) should be described as a percentage of specimen length.

To illustrate its use in a hypothetical example, the Vp_{init} value of a rock specimen of length 0.1m (L) is 3285ms^{-1}. Following 20 cycles of freezing and thawing a Vp_{20} value of 2915ms^{-1} is recorded. Since all velocity measurements in this research are undertaken on oven dried specimens, it is assumed any void is air-filled, and thus Vp_{air} is 330ms^{-1}. The value of w is calculated as 0.00142m. If the specimen length (L) is 100mm, IF_p is found from:

$$IF_p = \frac{w}{\text{Specimen length}} * 100 = \frac{1.42\text{mm}}{100\text{mm}} = 1.42\% \qquad [3.15]$$

Thus, the Index of Fracture Porosity represents the aggregate percentage volume of new voids introduced into a rock as a result of induced deterioration. This new void volume might be represented by a single fracture, or more likely, by a number of proportionately smaller fractures, microcracks and pores. Since it is unlikely that all induced fractures will actually lie perpendicular to the axis of wave generation the index value should only be regarded as a comparative indicator of induced fracture porosity. Because of the nature and direction of wave movement, it is also only applicable to compression wave velocity.

To illustrate the fact that the Index of Fracture Porosity takes account of the pre-test condition of the rock, an example is described here and compared with ID calculations obtained. Two contrasting rock specimens of length 100mm are subjected to experimental weathering. One is a weak, low density material with a pre-test P-wave velocity of 2500ms^{-1}, the other is a much denser, more competent material with a pre-test P-wave velocity of 6000ms^{-1}. Following experimental weathering, the corresponding velocities are reduced to 1880 and 3230ms^{-1} respectively, but in both cases, the calculated increase in fracture porosity (IF_p) is identical at 5%. In other words, the calculated amount of new void induced in the two rocks is identical, but the corresponding ID values are 25% and 46% respectively.

In practice, the time average formula [equation 3.13] presented by McDowell (1993) does not work well for vugs and air-filled cracks (Wyllie et al 1958) because they tend to act as an acoustic barrier and cause much of the incident energy to be reflected back. This means that, used in its original form, it could over-estimate Vp_1. However, used in the form presented here this problem is reduced because Vp_n (Vp_1) is not calculated, but measured.

Cracks which form parallel to the core axis will tend to produce a greater difference between Vp_{init} and Vp_n, exaggerating values of IF_p, than those which form perpendicular to it. This might also occur where the generated pulse intersects isolated cavities or other significant anomalies in the rock. In this experimental work the problem has been limited as much as possible by preparing specimens with the dominant alignment of bedding and other linear features perpendicular to the core axis.

3.4.3.3 Dynamic modulus of elasticity

Measurements of sonic velocity were used to find the dynamic modulus of elasticity (E_{dyn}) as described in ASTM D2845 -90:

$$E_{dyn} = \frac{\left[\rho Vs^2\left(3Vp^2 - 4Vs^2\right)\right]}{Vp^2 - Vs^2} \qquad [3.16]$$

Where ρ = rock density in kg/m^3, and E_{dyn} is given in GPa.

E_{dyn} can also be found from Vp and ν (Poisson's Ratio), but this requires estimating a value for ν which can lead to significant inaccuracies (McDowell 1993). The percentage change in E_{dyn} can then be used as an alternative indicator of durability. The IF_p, derived only from P-wave velocity, provides a useful measure of void introduced into the rock due to weathering, while percentage change in E_{dyn}, based on both P- and S-wave velocity and rock dry density, reflects the deterioration in elastic properties.

Several attempts have been made to correlate dynamic modulus of elasticity (E_{dyn}) with the static modulus (E_{st}). The two parameters should only be compared when loading rate and applied stress are the same (Whiteley 1983). Eissa and Kazi (1988) found the following empirical relationship:

$$\log_{10} E_{st} = 0.02 + 0.77 \log_{10} (\rho\ E_{dyn}) \qquad [3.17]$$

which gives a correlation coefficient of 0.96. Siggins (1993) suggests that E_{dyn} is generally around 30% higher than E_{st}, though agreement between them increases with higher values of elasticity (Eissa and Kazi (1988). McDowell (1993) offers a rule of thumb that E_{st} is likely to be about 15% of E_{dyn} where the Velocity Index is less than 0.6.

The ASTM standard D2845 (1990) states that elastic constants should not be applied to rocks with a degree of anisotropy exceeding 6% as determined from compression wave measurements in orthogonal directions. It was not possible to measure the anisotropy of the specimens used in this work due to their geometry, and therefore the extent of error in absolute values for E_{dyn} is unknown.

3.4.3.4 Pulse velocity measurement method

Acoustic wave propagation velocity, necessary to determine any of the indices discussed in section 3.4.3, can be determined from both bar resonance and ultrasonic pulse methods (ISRM 1978a). The latter has been used throughout this experimental work (refer to Figure 3.7). In this, a source piezoelectric transducer transmits a pulse through the axis of a specimen to a receiving transducer. Both P- and S-wave transducers were centred at a frequency of 29kHz with a usable range of 18 to 65kHz, and were housed in a steel casing to reduce any stray electromagnetic field. An Oyo pulse generator unit, model 5217A, was used to drive the transmitting transducer, providing a 500V amplitude 10μs pulse, repeated at 64 or 128 cycles per second. An Oyo Sonicviewer oscilloscope was used to measure the pulse travel time, and was powered with a 12v battery and charging unit.

The bottom of the rock specimen was placed on the receiving transducer with the transmitting transducer placed at the other end. Good coupling was ensured by adding a 6.6kg weight, giving an approximate normal stress of 34kPa, or 0.034N/mm^2. Vaseline coating of transducer platens was also used to improve coupling for measurements of Vp in accordance with manufacturers advice. The apparatus was set up on a firm, flat surface.

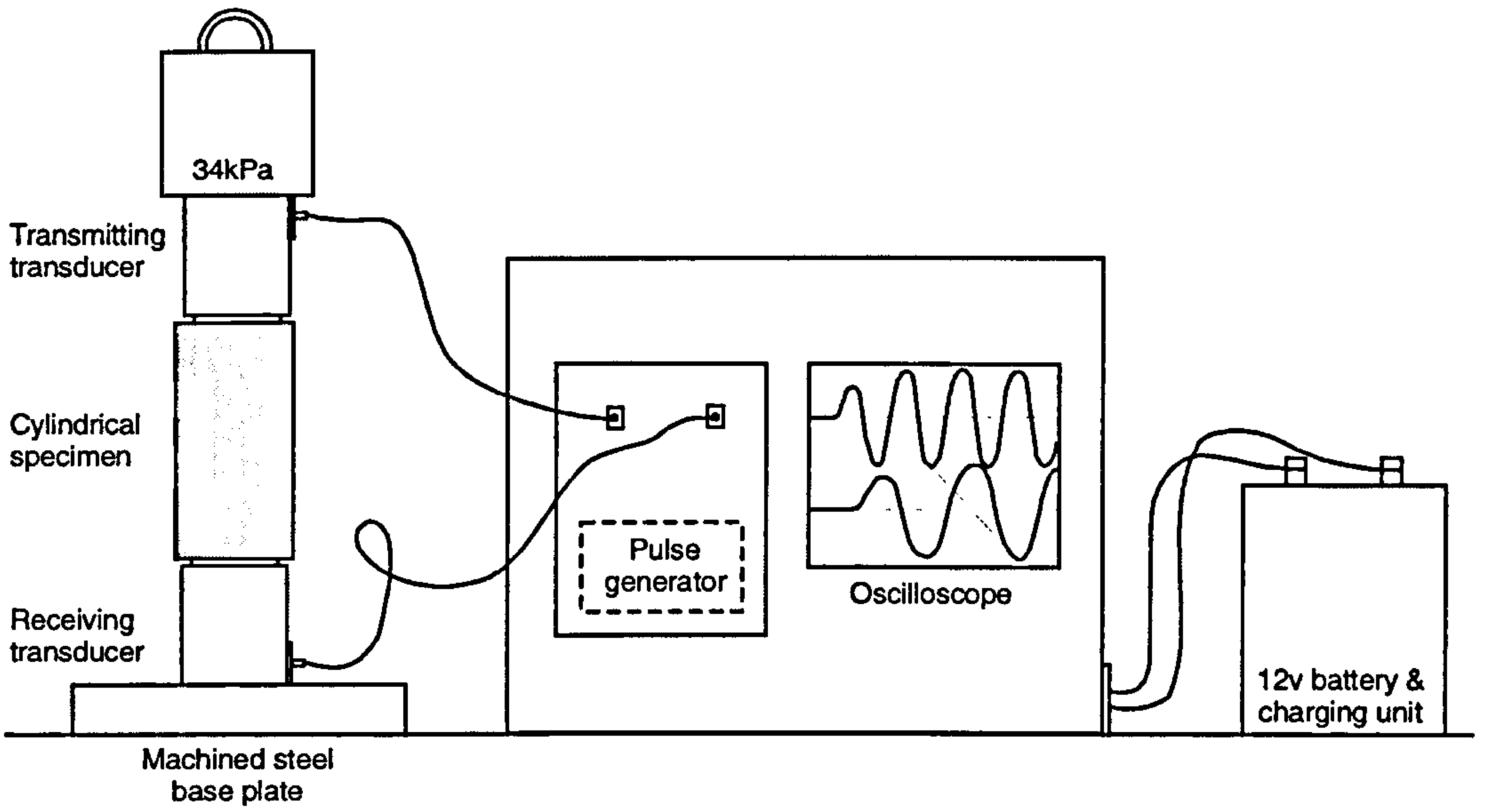


Figure 3.7 Schematic representation of ultrasonic pulse velocity measurement apparatus

Prior to measuring sonic velocity, oven-dried specimens were cooled at room temperature and tested within one hour to reduce the possible effect of moisture uptake from the atmosphere. Pulse travel time was determined from the mean of a minimum of three measurements of each specimen, using more if necessary to achieve greater consistency of readings. Travel time was defined following manufacturers recommendations, in which P-wave velocity was based on the 'first break', and S-wave velocity was based on phase difference. The first phase was determined for S-wave velocity through the rock specimen, and the second phase was based on a standard first peak for the coupled S-wave transducers.

Velocity was then determined from:

$$Vp = \frac{L}{t * 10^{-6}} \text{ m sec}^{-1} \qquad [3.18]$$

Where L is the length of the specimen (m) and t is the transit time of the pulse (μs).

For measurement of ultrasonic pulse velocity, specimens should effectively be infinite in length in comparison to the wavelength (γ) of the generated pulse (ISRM 1978a). This can be achieved where the mean grain size is less than the pulse γ, which in turn, is less than the minimum specimen dimension (which for cylindrical cores is the diameter). The ISRM (1978a) recommend that pulse travel distance is at least ten times the diameter of the mean grain size, while the ASTM (1990) recommend a factor of fifteen. With a nominal specimen length of 0.1m used throughout this investigation, the mean grain size of all specimens lies well within the tightest of the criteria stated.

Both of the above standards also set out requirements for the ratio of specimen diameter (D) to pulse γ. The ISRM (1978a) require a ratio of ten, while the ASTM (1990) require a ratio of five. Siggins (1993) argues that in the light of a study by Blair (1990), these requirements can be relaxed such that $D/\gamma > 1$. This is because bar velocity (Vb) increases towards infinite body velocity (Vp) as the ratio of D/γ increases from 0 to 1. At values of <1, therefore, the velocities measured do not represent infinite extent and are thus subject to boundary effects of the specimen. Blair (1990) states that shear wave velocity is independent of the aspect ratio (L/D) in cylindrical cores and thus it could be argued that these standards do not apply and that S-wave velocity represents infinite specimen length regardless of the D/γ ratio.

However, closer inspection of Blair's (1990) data, and data from other studies reviewed by Blair, indicates that with caution, values of $D/\gamma < 1$ could also be acceptable. For specimens with an aspect ratio of 2, which corresponds to that used in the current study, Vb approaches Vp at D/γ values exceeding 0.5.

Blair (1990) concludes that for this aspect ratio and a D/γ ratio of >1, the velocity measured will lie within 1% of actual P-wave velocity. Using an approximation of γ from the ratio of velocity to frequency, in this case 65kHz, and a specimen diameter of 0.05m, the criteria for $D/\gamma > 1$ are satisfied here for velocity measurements less than 3250ms^{-1}. For $D/\gamma > 1$ this means that some measurements could be subject to boundary effects (New and West 1980) causing an under-estimation of pulse velocity. This applies to SpaL and MetS, and to some individual specimens for OolL, HdCh, CalS and LamZ. For $D/\gamma > 0.5$, the criteria are satisfied for all velocity measurements less than 6500ms^{-1}. This means that a small number of individual specimens for SpaL could be subject to boundary effects.

3.4.3.5 Factors influencing wave propagation velocity

Work by New and West (1980) on the effects of discontinuities on wave propagation through rocks showed that the greater the acoustic closure, the higher the velocities achieved. In other words, the more open a joint is, the greater is the wave attenuation. The relationship between fracture width and sonic wavelength is also important, as is the nature of any infilling material (McDowell 1993). Under natural conditions, acoustic closure can be achieved by the load due to overburden, by rock bridges giving only partially open fractures, or by dense joint infilling. In the laboratory testing environment, closure can also be achieved by application of a dynamic or static load. Work by New (1976) showed that acoustic closure was possible with low normal stresses of 0.4MPa in chalk to 0.75MPa in sandstone, representing shallow depths of around 15 to 35m respectively. The purpose in this work of applying a normal load of 34kPa to specimens was to achieve good coupling with the transducers, but this load lies well below the load necessary to create acoustic closure.

3.4.3.6 Measurement accuracy and potential sources of error

Five potential sources of error in the measurement of acoustic wave velocities in stone weathering studies were identified by Murphy et al (1996): (a) intra-specimen variation (ie repeatability); (b) inter-specimen variation; (c) specimen geometry; (d) moisture content; (e) applied pressure. The potential effects of these factors are considered below:

Repeatability: The vast majority of travel time readings fell within an accuracy of ±0.5% which is comparable to that achieved by Allison (1988) using the Grindosonic resonance bar technique. Accuracy was lower at ±1% for the two strongest rocks, SpaL and MetS, a direct function of their very rapid travel times. This meant that variations of 0.1µs, the smallest recordable travel time unit, would have a larger effect on percentage variation.

Inter-specimen variation: Variation between specimens is expected even when they are derived from the same original block because even minor variations in weathering, microcrack density and porosity affect wave propagation. In fact specimen variation was deliberately incorporated into this experimental work. This is actually advantageous in many respects because it enables the causes of these variations to be investigated and conclusions drawn, although it has the disadvantage of producing large standard deviations about mean values.

Specimen geometry: All specimens except MetS (see section 3.6.2) were cut to an identical geometry to eliminate this potential source of error.

Moisture content: While S-wave velocity appears to be little affected, P-wave velocity generally increases with increasing moisture content (McDowell 1993). To remove any potential source of error associated with moisture content, all specimens were oven-dried and room cooled prior to measurements being taken. To determine the potential for moisture uptake from the atmosphere, a set of control specimens was repeatedly measured over several hours on a humid day and some specimen surfaces were also variously wetted. The amount of moisture uptake, if any, was insufficient to cause any significant change in travel time readings.

Applied pressure: A standard normal load was applied for all measurements to remove variations in load as a potential source of error. The amount of load applied was insufficient to cause acoustic closure.

Other potential sources of error: In the early stages of testing, it was apparent that some variation of readings occurred if specimens, notably SpaL, were oriented differently. To remove this as a potential source of error, specimens were marked and subsequently oriented the same way for all further measurements. Transducers were also placed in a fixed position relative to the specimens for each measurement.

3.4.4 Qualitative records of deterioration mode

Before and after experimental weathering and at the intervals stated above, a detailed, annotated pictorial record was made for each specimen. A selection of colour photographs was also taken to provide evidence of particular features of interest. The object of keeping a pictorial record was to describe, in qualitative terms, macro changes in the appearance of specimens and the relationship of these changes to rock flaws recorded prior to testing. Thus the pattern and location of individual fractures was recorded, as well as areas of scaling, the changing form of specimens as material was removed, areas of granular disintegration, and the like.

3.5 Measurement of Other Rock Properties

Most measurements made of rock properties were based on standards published by the International Society for Rock Mechanics (ISRM) (Brown 1981, published for the Commission on Testing Methods) and the American Society for Testing and Materials (ASTM). Other methods and standards used are cited as appropriate. The methods selected are described below.

3.5.1 Dry density (ρ), effective porosity (n_e) and water absorption capacity (W_{ab})

In order to determine these properties, three quantities were required: dry mass (M_d) or grain mass; saturated mass (M_s); and volume (V). Specimen dry mass (M_d) was obtained following a period of oven drying at 105°±2C to constant weight. Constant weight was deemed to have been achieved when less than 0.2% variation occurred over a 24 hour period. A period of 72 hours was generally sufficient to achieve drying, but some of the denser rock types required an additional 24 hours. Specimens were cooled at room temperature prior to weighing. Weighing accuracy was as for weight loss (section 3.4.1).

Saturated mass (M_s) was measured following a period of free saturation (ie at atmospheric pressure) to constant weight as defined above. Specimens were surface-dried using a damp chamois leather, which due its low absorbency, ensures water is not drawn out of the specimen.

Volume (cm^3) was determined on saturated specimens in one of two ways, the choice dependent on the size of specimen. The 'displacement method' (Brown 1981) was used for specimens exceeding 300g dry mass, while the 'buoyancy method' (Brown 1981) was used for specimens less than 300g dry mass (the slake durability cubes only). These choices were simply a function of the balance equipment available. In the displacement method, the specimen was suspended in a bowl of water (with an identical water temperature to that from the bath), using a fine mesh net. The bowl was placed directly on top of the balance. The increase in mass recorded on the balance was termed mass displaced (M_{disp}), and is equal to the volume of the water displaced. In the buoyancy method, the specimen was suspended *beneath* the balance such that it, and the mesh net holding it, were completely submerged in a bowl of water. The mass recorded, termed mass submerged (M_{sub}), was subtracted from M_s to give volume, V. In this case, reduction in mass supported by the balance between M_s and M_{sub} is equal to the volume of water displaced. On obtaining M_{disp} or M_{sub} for each specimen, it was replaced in the water bath and re-saturated prior to re-weighing for M_s. Variation of water density (ρ_w) from 1000g/ml was negligible over the range of water temperatures concerned and has therefore been disregarded in all of the above calculations. All measurements of mass are in grams.

The dry bulk density (ρ) was determined from:

$$\rho = \frac{M_d}{V} \qquad (g/cm^3) \qquad [3.19]$$

Effective porosity (n_e), the volume of connected pores as a percentage of the bulk volume of the specimen, was determined from:

$$n_e = 100 * \frac{M_s - M_d}{V} \qquad \% \qquad [3.20]$$

Water absorption capacity (W_{ab}), the weight of the water in connected pores as a percentage of the bulk weight of the rock, was determined from:

$$W_{ab} = 100 * \frac{M_s - M_d}{M_d} \qquad \% \qquad [3.21]$$

The ability of a porous material to absorb moisture is of considerable significance in determining its weathering susceptibility. The method of free saturation used here means that $W_{ab} \times \rho = n_e$, and as such the two properties W_{ab} and n_e are closely related.

3.5.2 Total porosity (n_t) and saturation coefficient (S)

Using ambient pressure saturation it is likely that the finest, connected pores will remain unfilled with water. Vacuum saturation was therefore used to obtain a value for maximum connected porosity. This value is termed *total connected porosity* (n_t). In fact <u>true</u> total porosity (ie including unconnected pores) can only be obtained satisfactorily by obtaining grain volume (involving crushing the rock). The free saturation method has been used throughout this research because the data obtained are considered to be more representative of natural conditions.

Saturation coefficient is the ratio of rock moisture content ($M_s - M_d$) after free saturation, to the moisture content ($M_{s\ max} - M_d$) after vacuum saturation. It therefore provides an index of the proportion of pores which are accessible and likely to be water-filled under saturation at atmospheric pressure. Saturation coefficient is sometimes defined by others as the ratio of effective to total porosity (ie including unconnected pores), but since unconnected pores are never likely to become saturated this value is not particularly useful in a rock weathering study.

Vacuum saturation was obtained by immersing specimens in de-aired, distilled water at a vacuum of up to 80mb for a period of 24 hours.

3.5.3 Mercury intrusion porosimetry

Mercury intrusion porosimetry is based on capillary law which states that intrusion into pores of a non-wetting liquid such as mercury, depends upon the amount of pressure applied to that liquid. When such pressure is applied, the amount of intrusion which results at any given pressure is related to the diameter of pore throats by the Washburn Equation:

$$D = \frac{-4\gamma\cos\theta}{P} \times 10^6 \qquad [3.22]$$

Where D = pore throat diameter (μm), P = applied pressure of intruding mercury (N/m^2), γ = surface tension (N/m), and θ = contact angle. As pressure is increased, mercury intrudes into pores of smaller diameter.

Mercury intrusion porosimetry was undertaken using an Autopore II 9220 porosimeter, capable of delivering a maximum pressure of 414MPa (60,000psi) and of measuring pore throat diameters ranging from 0.0036μm to 100μm. Measurements are based on the assumption of cylindrical pores, with a standard contact angle of 140° and a mercury surface tension of 485d/cm. Specimens were nominally $3.0cm^3$ and a single test was performed for each value obtained. While the high pressure impregnation of the fluid medium indicates that a maximum effective porosity should be obtained using this technique, there are also indications (discussed in Chapter Five) that the viscous nature of the fluid compared to water renders the pores less accessible to mercury. For this reason, it was deemed inappropriate to use the porosity value obtained to determine saturation coefficient. Mercury intrusion porosimetry was used to obtain porosity (n_m), pore size distribution, modal pore diameter and microporosity (μn_m). The latter is defined as the percentage of pores ≤ 1μm in diameter (after McGreevy 1982, 1996).

3.5.4 Compressive strength (C_o), Modulus of Rupture (T_{mr}) and Point Load Strength (IS_{50})

Three strength tests were conducted: compressive strength (C_o), modulus of rupture (T_{mr}) and point load strength (IS_{50}). Published standards recommend between 5 and 10 test pieces are used for each of these tests. The actual number of pieces used is given in Table 3.3. For the freeze-thaw test only, point load tests were conducted prior to experimental weathering (denoted 'pre' in Table 3.3) and at intervals throughout testing ('c' indicates the number of cycles passed at the time of point load testing). Similar test procedures were undertaken for T_{mr} for HdCh and LamZ only.

The tensile strength of samples was obtained using the three point disc test, described in Brook (1990 and 1993) and ASTM C-99-87 (1987). This utilises the equation:

$$T_{mr} = \frac{3PL}{2d^2b} \qquad [3.23]$$

Where P = load at failure (kN); L = span (mm) of disc between supports; d = mean depth of specimen cross section (mm); and b = breadth (diameter in mm) of specimen. The value of $0.5T_{mr}$ represents a very close approximation to T_o (tensile strength) (Brook 1990).

Compressive strength and point load strength were measured in accordance with the requirements of Brown (1981) and Broch and Franklin (1972) respectively. For the latter, all tests were normalised for a 50mm diameter specimen to give IS_{50}.

	LdCh	MagL	OolL	HdCh	SpaL	WeaS	WeaSw[1]	CalS	MicS	LamZ	MetS
C_o	10	12	8	8	8	12	n/a	8	10	9	n/a
T_{mr}	17	8	8	Pre = 9 5c = 7 8c = 5 15c = 2	8	14	n/a	14	8	Pre = 8 5c = 7 8c = 6 15c = 3	9
IS_{50}	Pre = 10 6c = 8 14c = 5 18c = 6	Pre = 8 12c =8 20c = 6 32c = 6	Pre = 10 11c = 3	Pre = 8 5c = 12 8c = 14 15c = 14	Pre = 8 25c = 8 47c = 8 75c = 8	Pre = 8 12c = 8 20c = 8 33c = 8	Pre = 6 12c = 6 20c = 6 33c = 5	Pre = 10 11c = 10 21c = 9 30c = 9 41c = 8	Pre = 10 12c = 8 20c = 8 32c = 4 45c = 4	Pre = 11 5c = 3 8c = 16 15c = 9	Pre = 8 25c = 8 47c = 8 75c = 8

Table 3.3 Number of specimens for mechanical tests[2]

Note 1 WeaSw refers to an extra set of particularly weathered specimens of WeaS.
Note 2 See text for explanation of codes.

3.5.5 Elasticity

Since dynamic elasticity, derived from sonic velocity measurements, is used in this research as both a rock property and an indicator of deterioration, its measurement has already been described in section 3.4.3.3.

3.5.6 Petrographic analysis and mineralogical composition

Thin sections were analysed in accordance with the ISRM suggested method (Hallbauer et al 1978), using a cross-polarising petrographic microscope and point counter. A record was made of composition, microfractures, alteration, fabric and grain size.

3.5.7 Scanning electron microscopy

Scanning electron microscopy was undertaken using a Cameca SX-50 microprobe. Pre and post-test analysis of specimens was undertaken for the freezing and thawing and salt weathering tests only, using a single test specimen in each case. Test pieces were cut from specimens which were considered to be representative of the main sample in terms of deterioration severity and mode. Test pieces were unpolished and untreated except for a gold coating and analyses were undertaken using both secondary and backscattered electron image modes.

3.6 Rock Sampling and Preparation

3.6.1 Selection of rock types

Rock types and suitable sampling sites were selected to satisfy several criteria:

(i) *Rock variety:* Rock types were selected to represent a wide range of materials in terms of their lithological and mechanical properties (eg strength, fabric, texture, weathering grade, heterogeneity and anisotropy). Notwithstanding these criteria, very weak rocks were disregarded as they were likely to deteriorate so severely and rapidly as to prevent collection of useful data. Similarly, very strong rocks were disregarded because significant deterioration would be unlikely to occur over the short duration of the experimental programme.

(ii) *Practical considerations:* Sites from which samples were obtained had to be safe, with reasonable access. Loose blocks, either on or at the foot of the slope, of an appropriate size and shape for transport and preparation, were required.

(iii) *Type of source slope:* The vast majority of slopes investigated in this research (Part Two) were either quarries or road cuttings; laboratory samples were selected from a representative range.

Of the ten rock types tested, five were calcareous and five were arenaceous. Four rocks were obtained from road cuttings (MagL, WeaS, CalS, MetS) - one of which (MagL) was under construction; four were obtained from disused quarries (LdCh, OolL, SpaL, MicS), one of which (LdCh) is also a road cutting; and the remaining two were taken from an active quarry (LamZ), and a semi-active quarry (HdCh). The selection of blocks from each of the source sites was undertaken based on careful observation and description of the slope and its material. Blocks which displayed similar characteristics to the in situ material were selected, although as loose blocks, they might have been subject to more intense weathering, and might therefore, be less durable. Alternately, they might represent slightly more durable rock from the slope.

3.6.2 Sample collection, storage and preparation

Following collection from the source site, samples were transported to the Engineering Geology laboratories at the University of Leeds and stored temporarily at room temperature prior to preparation and testing. Following rough trimming of sample blocks using a diamond saw with water coolant, individual test specimens for freezing and thawing, salt weathering and wetting and drying tests were cored as cylinders of nominal size 100 x 50mm diameter using an industrial corer with diamond bits and water coolant. By maintaining a common size and shape for all specimens, the effect on test results due to such variations (Goudie 1974) was eliminated. The exception to this rule was the metasediment (MetS) which proved difficult to core. Rectangular block specimens were therefore saw-cut with dimensions 100x50x50mm. After preliminary oven drying, the ends of the cylinders were ground to form a smooth and parallel finish. This was necessary for sonic velocity measurements to be taken. For slake durability testing, cubes of nominal side length 30mm were cut with all corners and edges trimmed using a diamond saw. The size of these cubes was varied slightly to achieve the total and individual specimen masses recommended by Franklin and Chandra (1972). Because of specimen geometry, it was not possible to measure fracture density or sonic velocity for these test specimens. It is possible that the preparation procedures for all test specimens might have led to fracturing. However, since all were exposed to the same procedures, comparability between specimens should not have been affected.

For compressive strength testing, cylinders of 25mm diameter nominal size were prepared as above, the lengths cut to satisfy the diameter to length ratio of 1:2.5 to 1:3. For tensile strength testing, discs of 50mm diameter were cut from the cylinders, the thickness satisfying the span to thickness ratio of about 3 to 1 (Brook 1993). Rough cut blocks of 25mm nominal side length were prepared for point load testing. Other specimens were prepared as necessary for mercury intrusion porosimetry, scanning electron microscopy and thin section analysis as described in section 3.5.

3.7 Pre-test Sample Descriptions

Five arenaceous rocks and five calcareous rocks varying from Silurian to Cretaceous in age were used in this experimental work. They are described below in terms of their physical characteristics, and some basic rock properties are given in Table 3.4. Descriptions are based on hand specimen and thin section analysis.

3.7.1 Low density chalk (LdCh)

This is an Upper Cretaceous Chalk from the Lewes Nodular Chalk Beds, Lewes, Sussex (GR: TQ 425100). It is a heterogeneous, SOFT CHALK (after Mortimore and Fielding 1990) containing a wide range of pre-existing flaws including fossils, deformation structures (from syn-depositional turbation), stylolites, healed and incipient fractures, iron oxide weathering stains and rough-textured higher porosity zones with micro-cavities. The rock is formed from high purity calcite (98%) derived from planktonic organisms (eg coccoliths and foraminifera), though also with fossilised benthic organisms (eg *micraster* spps, *echinocorys* and *mytiloides*.). *Zoophycos* trace fossils are also present (Mortimer 1997).

3.7.2 Magnesian limestone (MagL)

This is a Permian dolomitic limestone from West Yorkshire (GR: SE 443436). It is a very fine, moderately weathered (using the classification of Moye 1955) crystalline MAGNESIAN LIMESTONE containing many small voids and irregular cavities up to 5mm on the surface, rare stylolites, discoloured bands and spots, and incipient fractures. During preparation some specimens broke along fractures revealing weathered surfaces.

3.7.3 Oolitic limestone (OolL)

This is a Jurassic limestone from Hovingham, North Yorkshire (GR: SE 675750). It is a medium to coarse grained (up to 1.5mm), slightly to moderately weathered, weakly cemented OOLITIC LIMESTONE, containing many fossils, shell fragments and large round cavities up to 15mm diameter and 3mm deep, rare incipient fractures and some discoloration. Some small voids are present due to the dropping out of individual ooids.

3.7.4 High density chalk (HdCh)

This is an Upper Cretaceous Chalk of the Flamborough formation from North Yorkshire (Wood and Smith 1978) (GR: TA 047612). It is a uniform, hard to VERY HARD CHALK (after Mortimore and Fielding 1990) containing large fossil fragments up to 25mm, stylolites, rare calcite veins, discoloration and isolated calcite-infilled voids. One specimen contained a 1mm wide calcite healed vein. The rock is formed from high purity calcite derived from planktonic organisms, but also with many fossilised benthic organisms.

3.7.5 Sparry limestone (SpaL)

This is a Carboniferous Scar Limestone from Faulds Brow, north Cumbria (GR: NY 303407). It is a strong, dense, highly fossiliferous (up to 3mm) SPARRY LIMESTONE containing many 1-2mm wide, closed, angular, persistent calcite veins, often accompanied by a discoloured 'shadow'. Also present are stylolites and deformation structures, both with associated discoloration. There is an absence of macro fractures. Large areas of iron oxide staining occur throughout the material with particular concentrations around healed discontinuities.

3.7.6 Weathered sandstone (WeaS)

This is a Carboniferous Millstone Grit from West Yorkshire (GR: SE 137552). It is a medium to coarse grained, clay and mica-rich weathered SANDSTONE with many discoloration bands and small mudstone clasts (up to 12mm). Rarely, cavities <1mm diameter occur. The rock consists largely of detrital quartz grains with occasional angular, authigenic quartz overgrowths. Mineralogical composition is 85% quartz, 6% plagioclase feldspar, 6% iron oxide minerals, 3% lithic quartz and a trace of mica. There is little matrix material present.

3.7.7 Calcareous sandstone (CalS)

This is a Jurassic sandstone from Sutton Bank, North Yorkshire (GR: SE 515827). It is a fine to medium grained, indistinctly laminated, CALCAREOUS SANDSTONE with alternating patches of calcareous and quartz rich matrix. The sandstone contains irregular, indistinct, thin laminae, and shell fragments and has a pocked surface with random, rounded cavities contained within it. Quartz and calcite grains are angular and contained within a fine matrix of mainly calcite. Angular quartz comprises 60% grains, with weathered calcite matrix forming the remainder. One specimen (number 1) was notably more weathered than was typical for the sample, and also contained a distinctive lens-like sedimentary fabric.

3.7.8 Micaceous sandstone (MicS)

This is a Triassic sandstone from the St Bees formation at Birkhams Quarry, west Cumbria (GR: NX 955154) It is an alternating mica-rich (fine grained) and quartz rich (medium grained) MICACEOUS SANDSTONE with thin laminations and cross laminae, the boundaries of fine and coarse bands coinciding with laminae. The sandstone contains calcareous nodules (up to several mm), small incipient fractures along laminae, and rarely, large mudstone clasts. The rock comprises rounded detrital grains with many angular authigenic crystals and biotite mica, with a great deal of intergranular debris, mainly composed of clay minerals.

3.7.9 Laminated siltstone (LamZ)

This is a Carboniferous Coal Measures siltstone from Wigan, Lancashire (GR: SD 552014). It is an extremely closely laminated (1-5mm) SILTSTONE and very fine sandstone with tight 'fold' hinges, truncated surfaces and overlapping structures. It contains numerous sub- and persistent incipient fractures associated with deformed and alternating lamination boundaries and is highly anisotropic. The rock has a very flaky surface with many microcracks. Mica flakes drape around quartz grains, and the matrix material is primarily composed of muscovite, elongated in the direction of the fabric. Laminations are evident at a microscale by the distinct layering of quartz and groundmass layers. There are some random altered zones. The rock is composed of 78% quartz and 28% groundmass. There is also a trace of calcite and 2% iron oxide minerals and altered or weathered zones.

3.7.10 Metasediment (MetS)

This is a Silurian turbidite of Bannisdale Slates from east Cumbria (GR: SE 555077). It is a slightly metamorphosed, very fine grained sandstone turbidite METASEDIMENT containing strong, faint laminations and many strong, incipient fractures, often with associated discoloration. The rock has a uniform grain size and some large, partially infilled cracks. These are generally regular, but sometimes occur in an *en echelon* form. The rock fabric is picked out by slightly weathered (discoloured) bands of material. The rock is 70% quartz and 30% muscovite. Some of the quartz is altered and weathered.

Sample	n_e	ρ	μn_m	F_D	C_o
Low density chalk (LdCh)	33.4	1.74	65.6	3.9	6.6 *(1.9)*
Magnesian limestone (MagL)	14.4	1.62	75.3	5.7	7.8 *(1.7)*
Oolitic limestone (OolL)	17.0	2.16	93.2	2.0	11.6 *(5.6)*
High density chalk (HdCh)	22.0	2.01	99.6	2.3	47.0 *(5.5)*
Sparry limestone (SpaL)	0.5	2.66	18.2	13.9	79.7 *(23.4)*
Weathered sandstone (WeaS)	10.8	2.23	19.8	0.0	13.8 *(2.4)*
Calcareous sandstone (CalS)	18.2	1.95	29.4	0.9	31.6 *(6.1)*
Micaceous sandstone (MicS)	15.6	2.09	35.4	0.7	41.6 *(19.2)*
Laminated siltstone (LamZ)	6.8	2.52	97.6	22.2	81.0 *(12.4)*
Metasediment (MetS)	1.4	2.65	0.0	28.2	140.0

Table 3.4 Mean pre-test rock properties for the samples tested.
n_e = effective porosity (%); ρ = dry density (g/cm^3);
μn_m = microporosity (%); F_D = fracture density (x10^3 mm^2/mm^3);
C_o = compressive strength (MPa) *standard deviation given in parenthesis.*

CHAPTER FOUR
CONTROLS AND INFLUENCES ON ROCK DETERIORATION SUSCEPTIBILITY

4.1 Introduction

In this chapter, the results of the experimental weathering programme are presented and discussed. First, the extent, or severity of deterioration is described in terms of the deterioration indicators introduced in Chapter Three. Then, the relationships between a range of rock properties and deterioration are explored.

4.2 The Severity of Deterioration

Changes in deterioration indicators as the weathering tests proceeded are presented in Figure 4.1 (a to j) as mean sample values. In figures 4.2 to 4.11 variations in deterioration indicators for each individual specimen, and also intra-sample variability is shown. These Figures should be referred to throughout the following sections. Key data are also given in Tables 4.1 to 4.4.

4.2.1 Freeze-thaw (Table 4.1)

Weight loss (Figure 4.1a): With increasing number of freeze-thaw cycles all samples experienced a loss in weight, the mean value of which varied from negligible to 84%, with higher values for calcareous rocks. For calcareous rocks, greater pre-test compressive strength correlated well with increased weathering resistance but this relationship is less apparent for arenaceous rocks. One group of rocks, LdCh (Figure 4.2a) and OolL (Figure 4.4a), disintegrated rapidly and severely, giving substantial weight losses comparable to those of Goudie (1974) for chalk. The most notable breakdown occurred in the early stages of testing particularly for LdCh and there was little variation between individual specimens for these rocks. In a second group, MagL (Figure 4.3a), HdCh (Figure 4.5a), CalS (Figure 4.8a) and LamZ (Figure 4.10a), moderate weight loss occurred and there was greater variability between specimens. In a third group, SpaL (Figure 4.6a), WeaS (Figure 4.7a), MicS (Figure 4.9a) and MetS (Figure 4.11a), weight loss was minimal and there was generally little variation between specimens. Most samples deteriorated more rapidly in the initial stages of testing, with OolL, HdCh and some individual specimens of LamZ being the exceptions and showing a more concave temporal trend. Overall, weight loss results indicate considerable variation in the severity of deterioration between samples.

Fracture density (Figure 4.1b): The pattern of fracture density results is not dissimilar to that of weight loss and indeed, if one anomalous sample (LamZ) is excluded from calculations, the coefficient of correlation between weight loss and fracture density for this test is 0.96 (Figure 4.12).

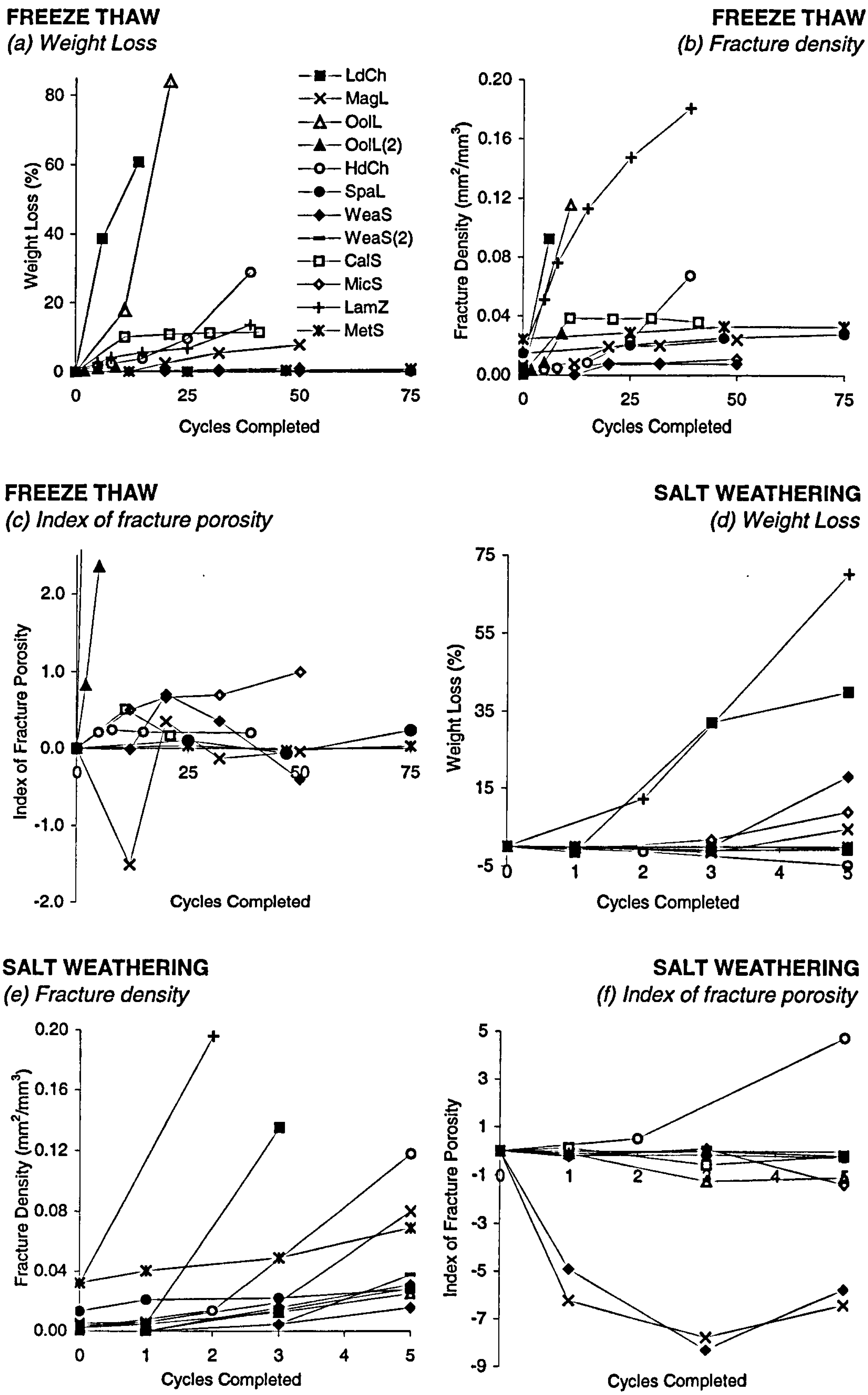
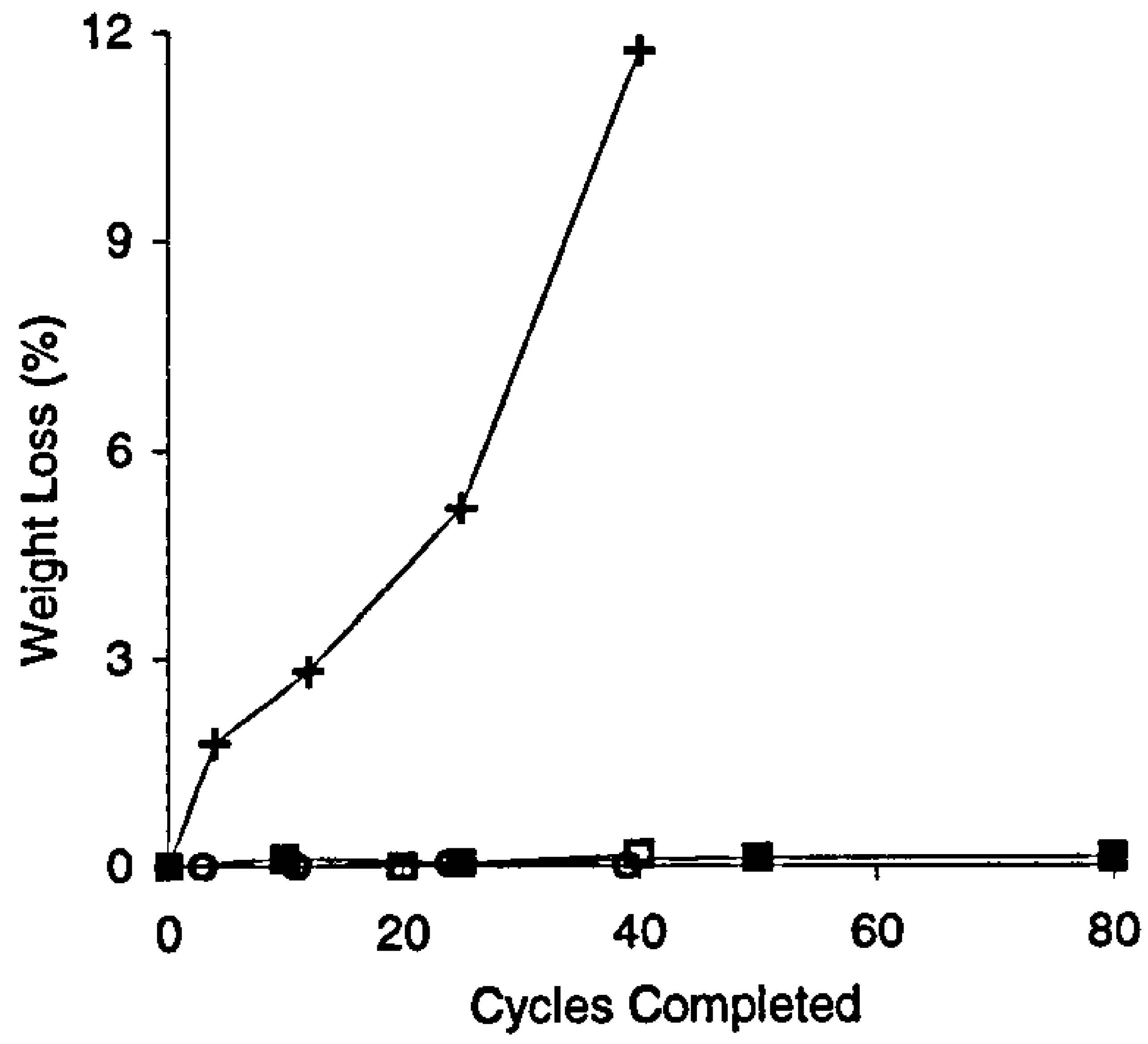
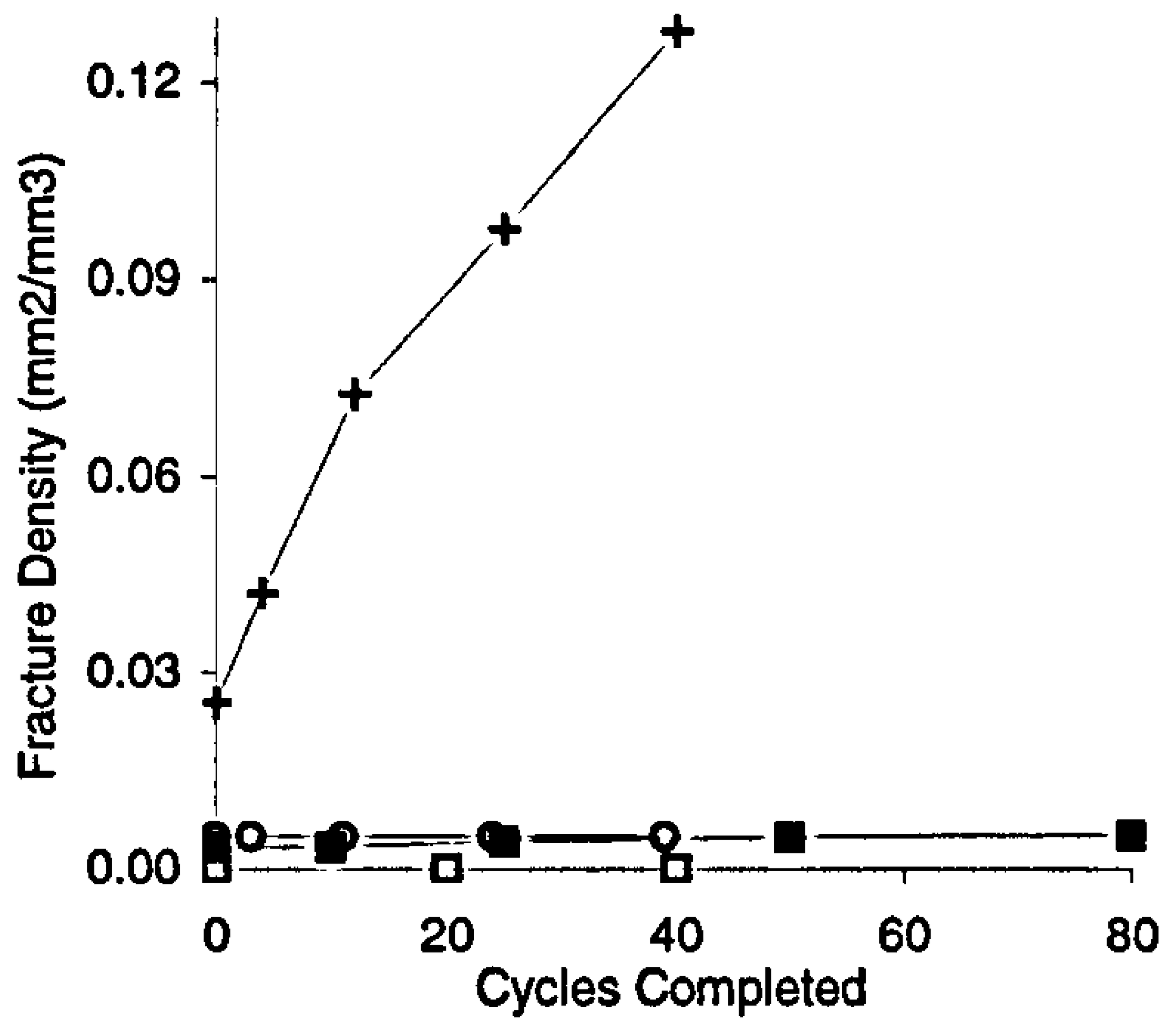


Figure 4.1 (a to j) Mean deterioration data for each test

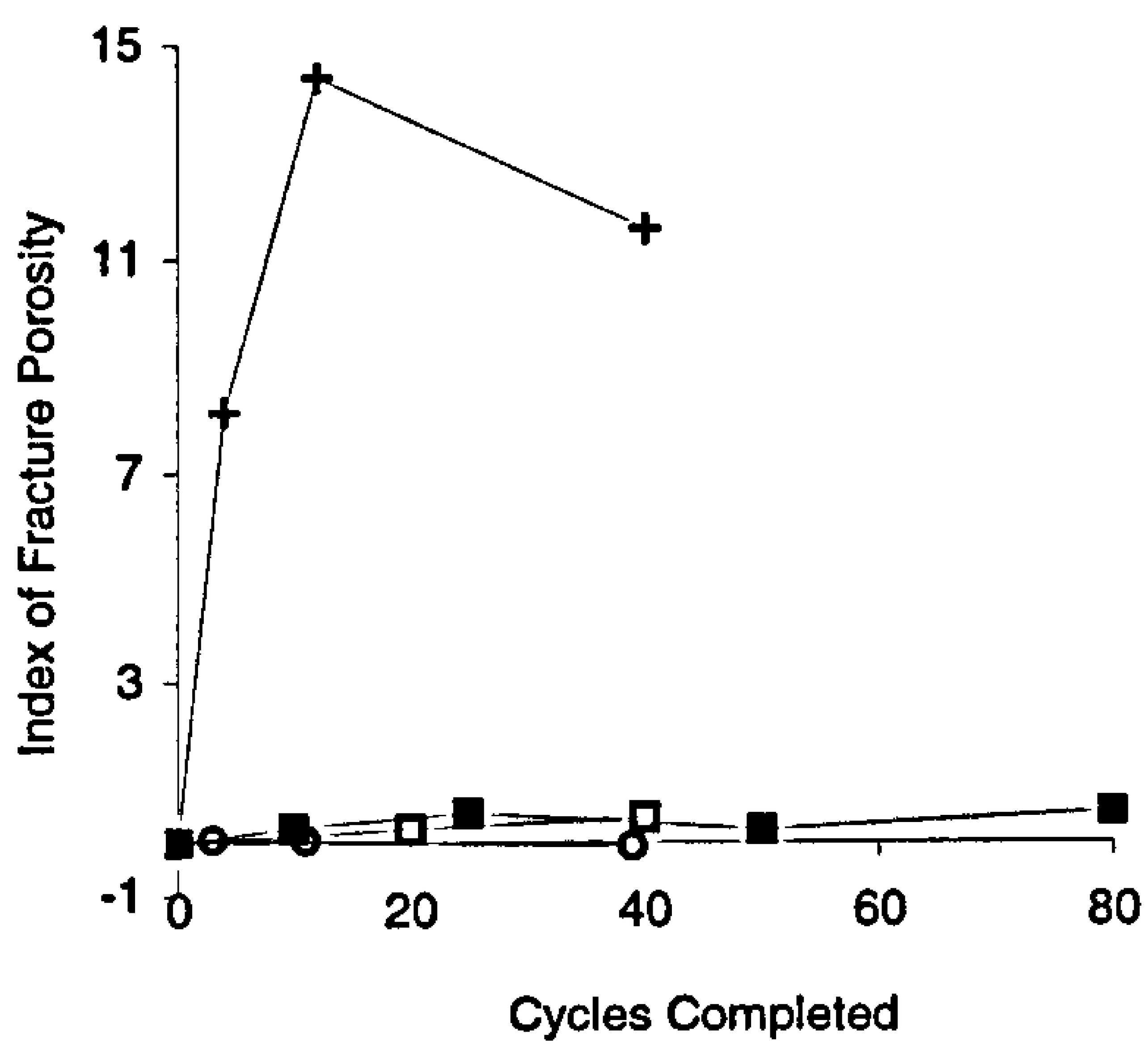
WETTING AND DRYING
(g) Weight Loss



WETTING AND DRYING
(h) Fracture density



WETTING AND DRYING
(i) Index of fracture porosity



SLAKE DURABILITY
(j) Retained weight

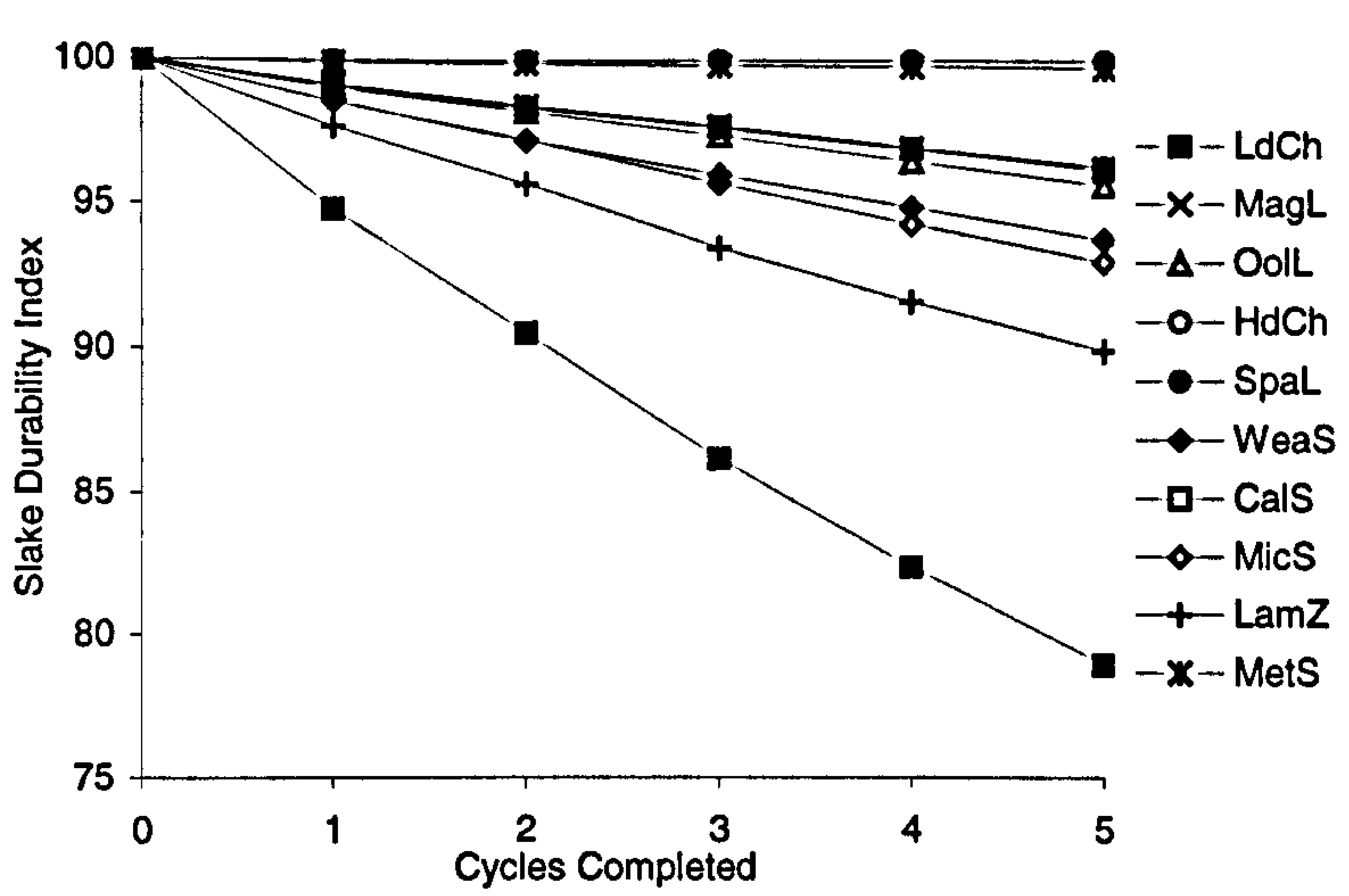


Figure 4.1 (g to j) Mean deterioration data for each test

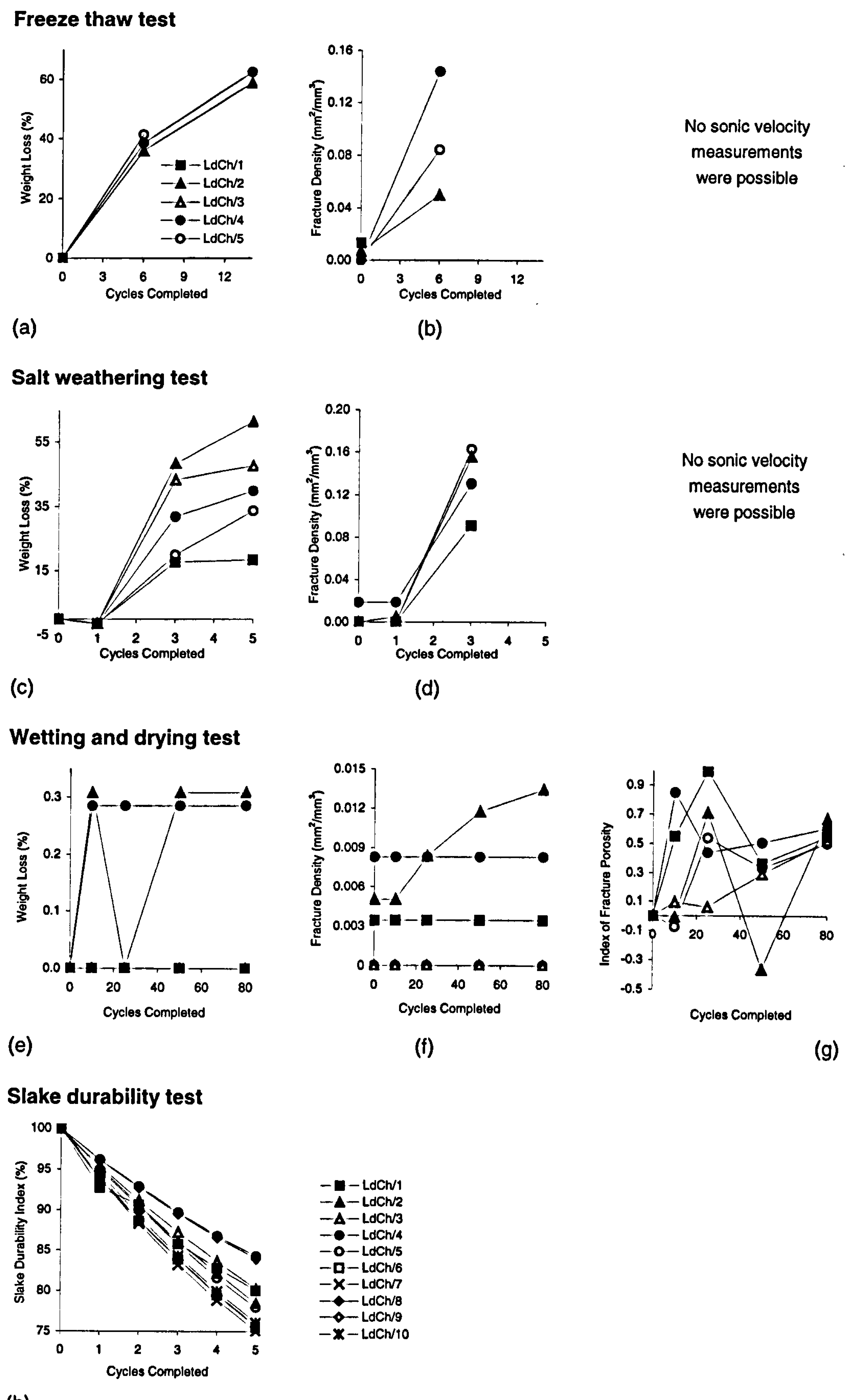


Figure 4.2 (a to h) Deterioration indicators for the low density chalk **LdCh**

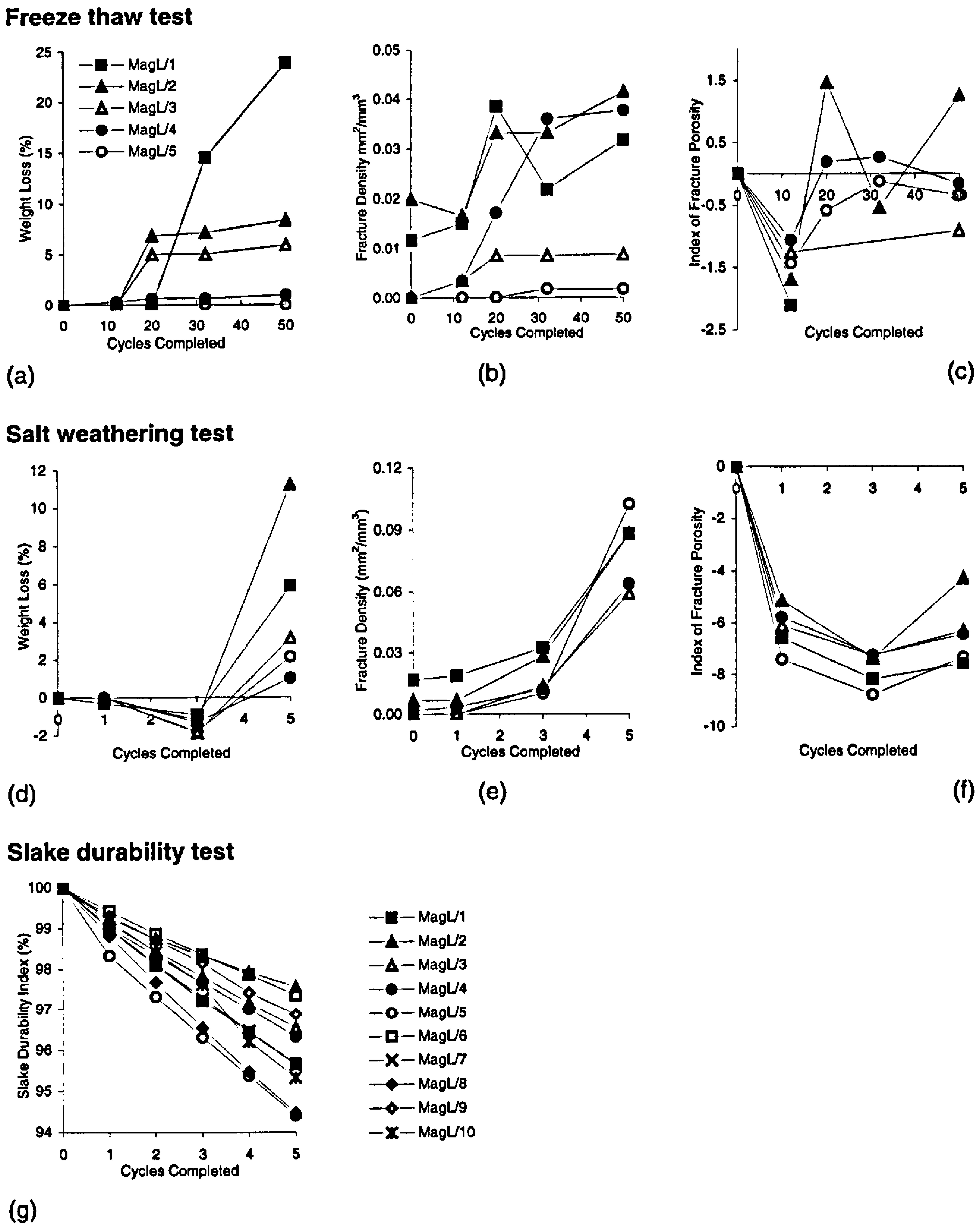
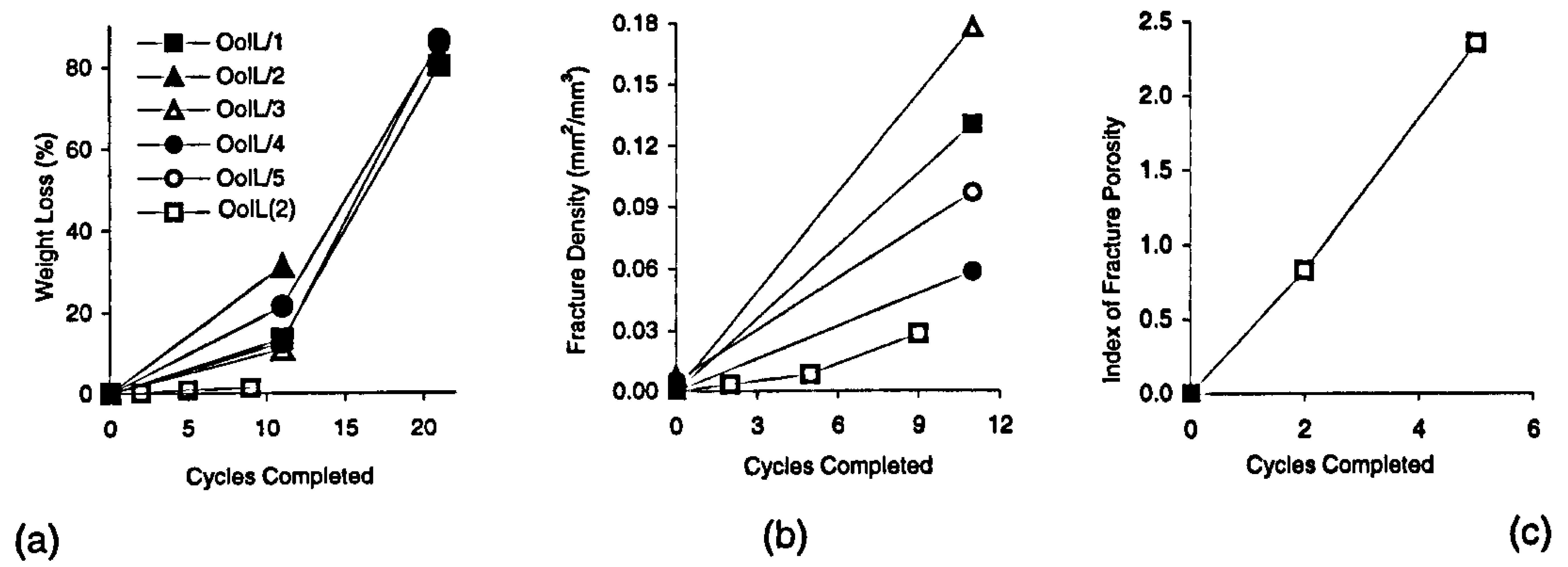
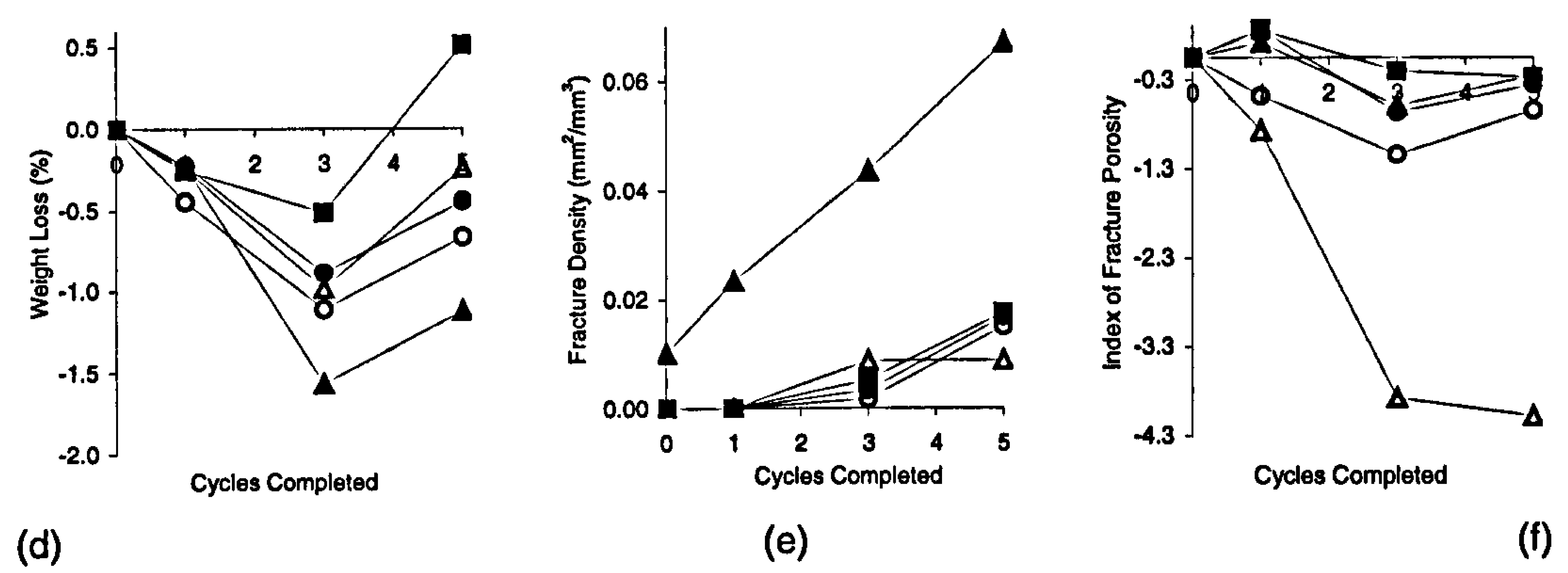


Figure 4.3 (a to g) Deterioration indicators for the magnesian limestone **MagL**

Freeze thaw test



Salt weathering test



Slake durability test

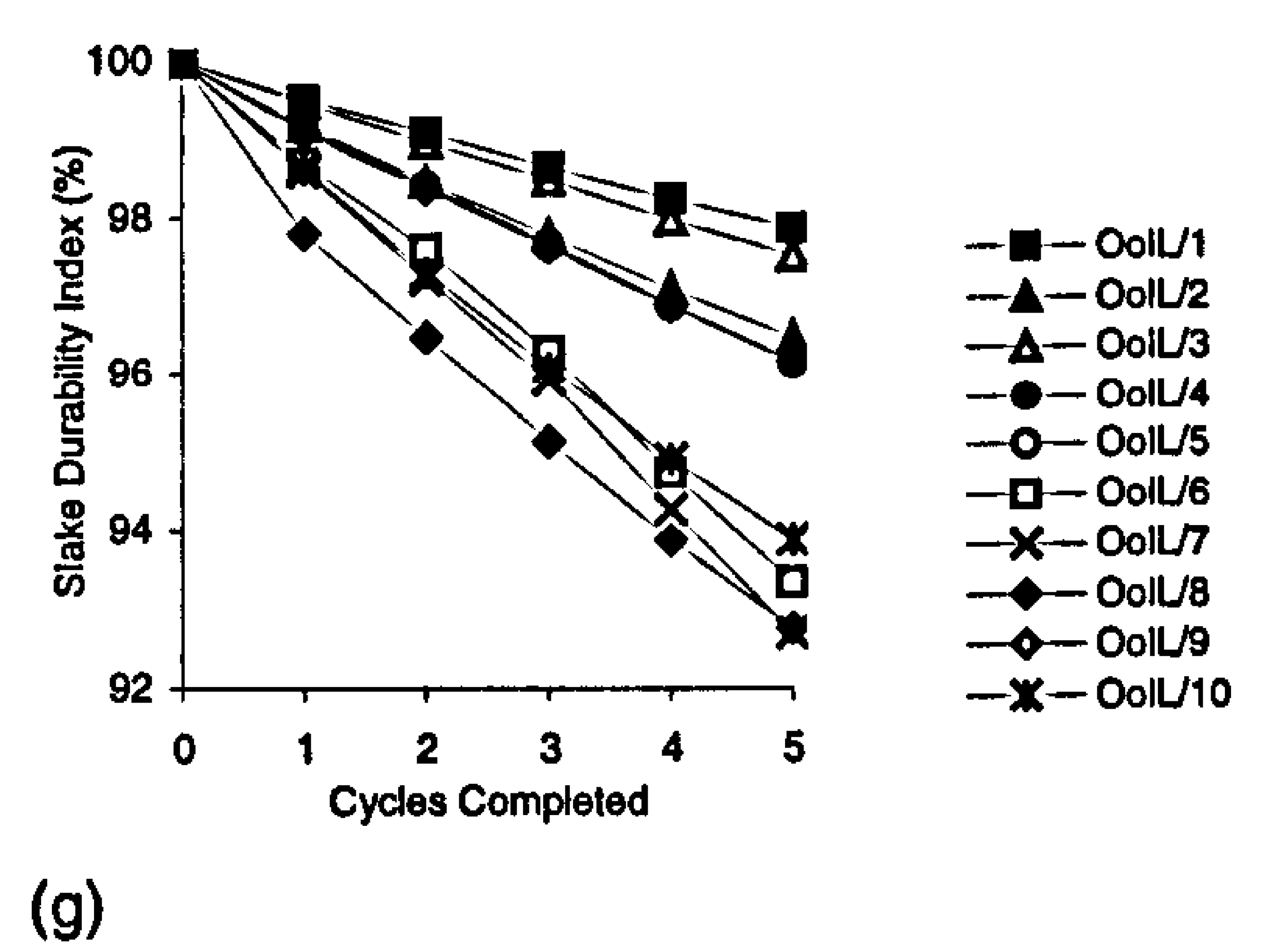


Figure 4.4 (a to g) Deterioration indicators for the oolitic limestone **OolL**

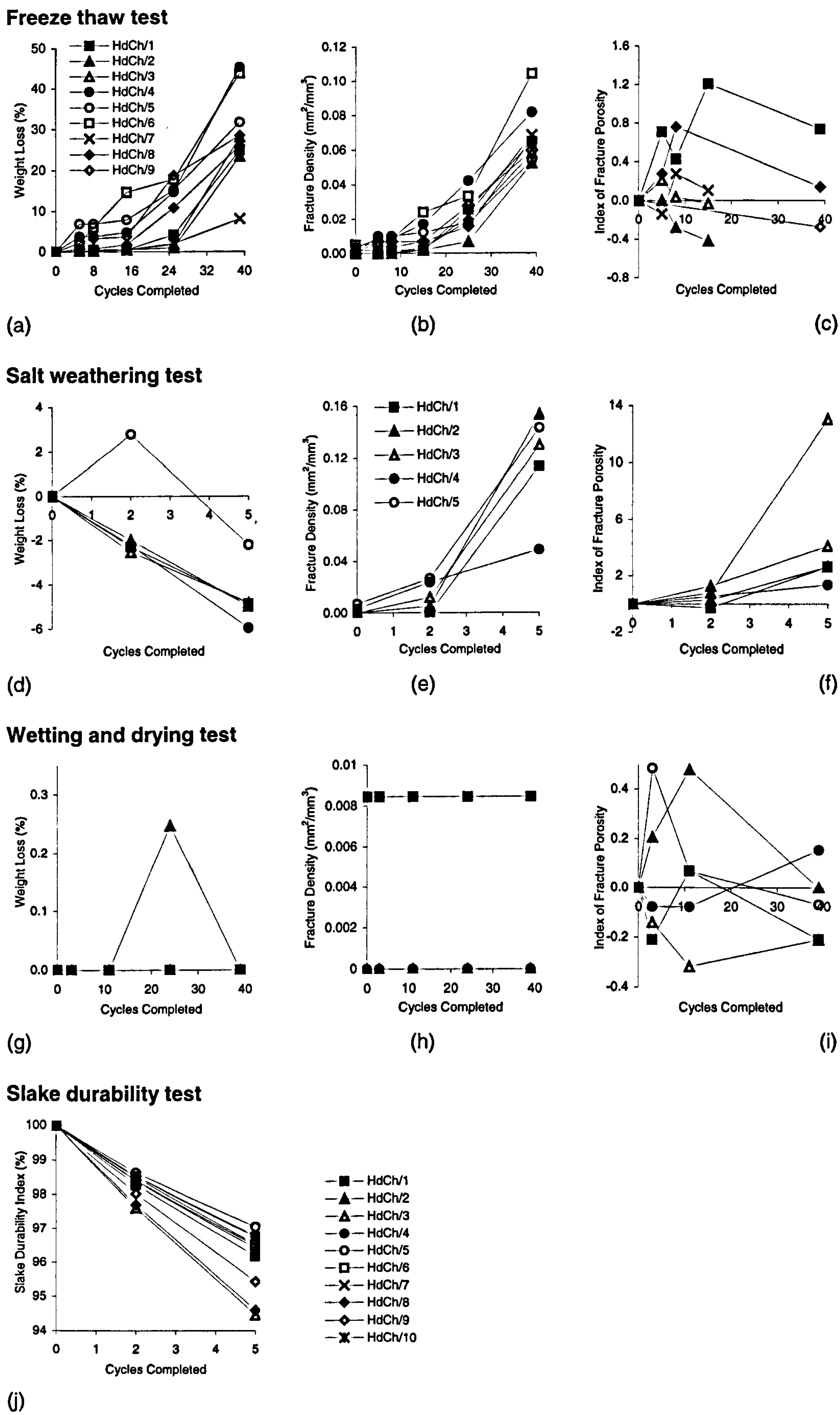


Figure 4.5 (a to j) Deterioration indicators for the high density chalk **HdCh**

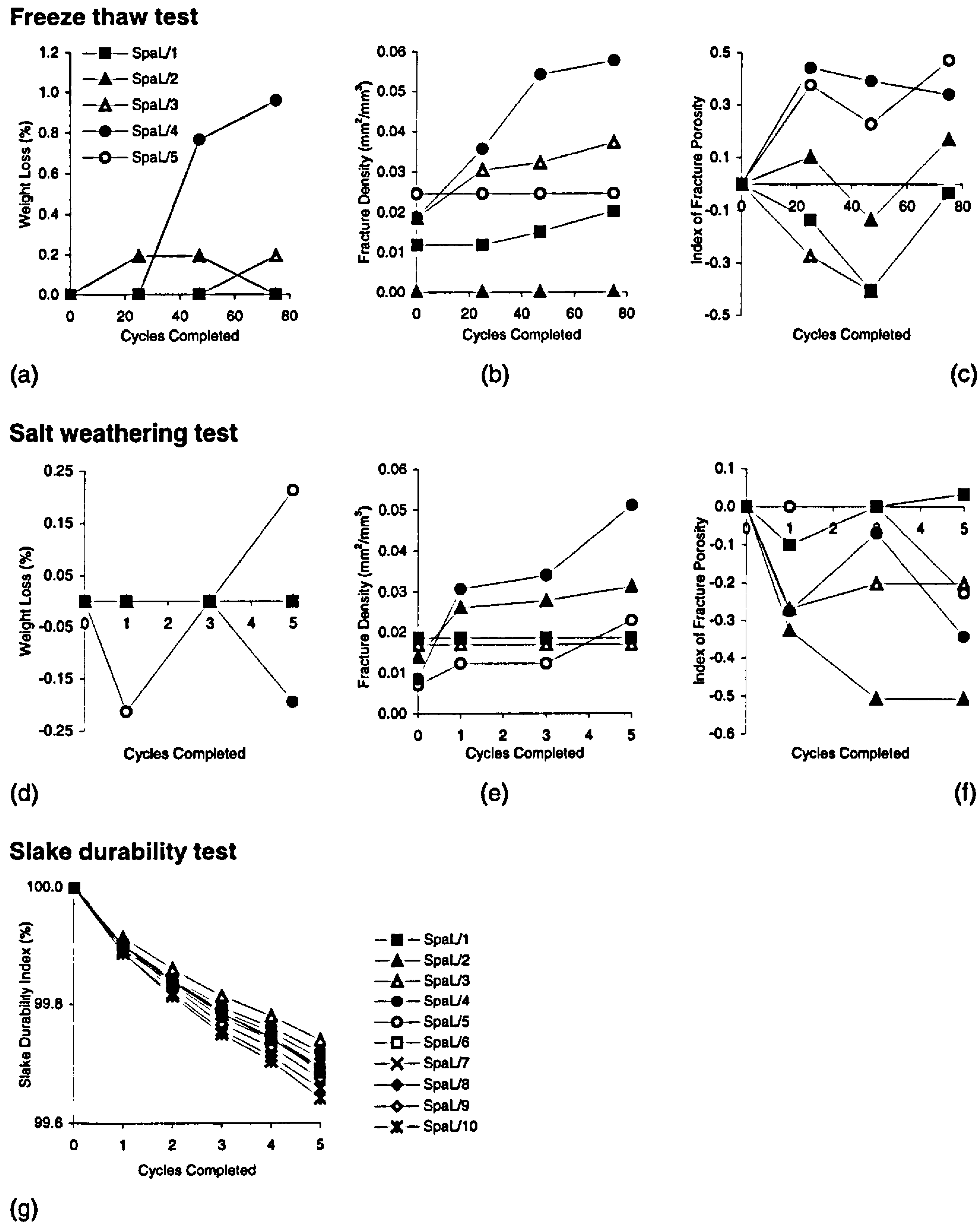


Figure 4.6 (a to g) Deterioration indicators for the sparry limestone **SpaL**

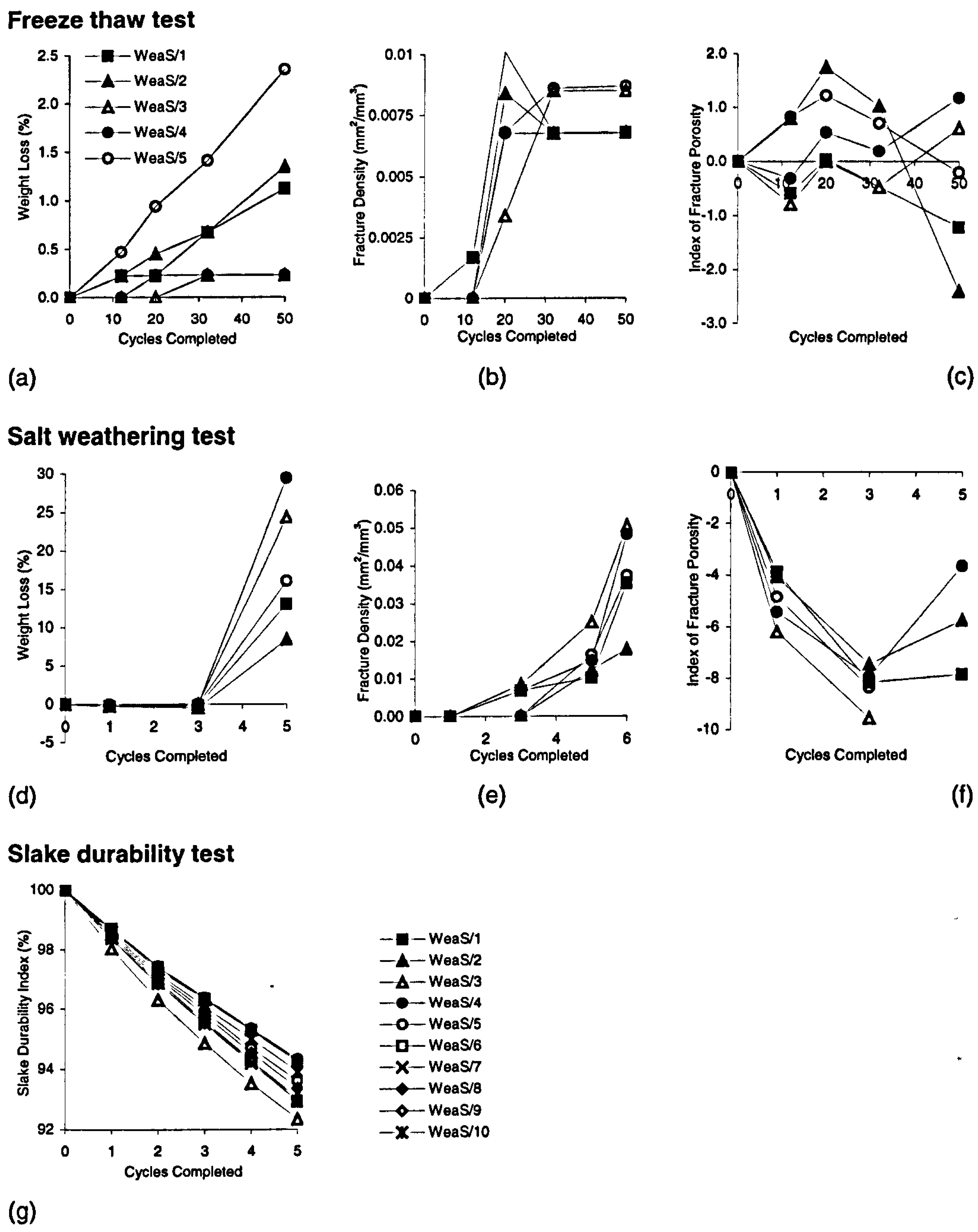


Figure 4.7 (a to g) Deterioration indicators for the weathered sandstone **WeaS**

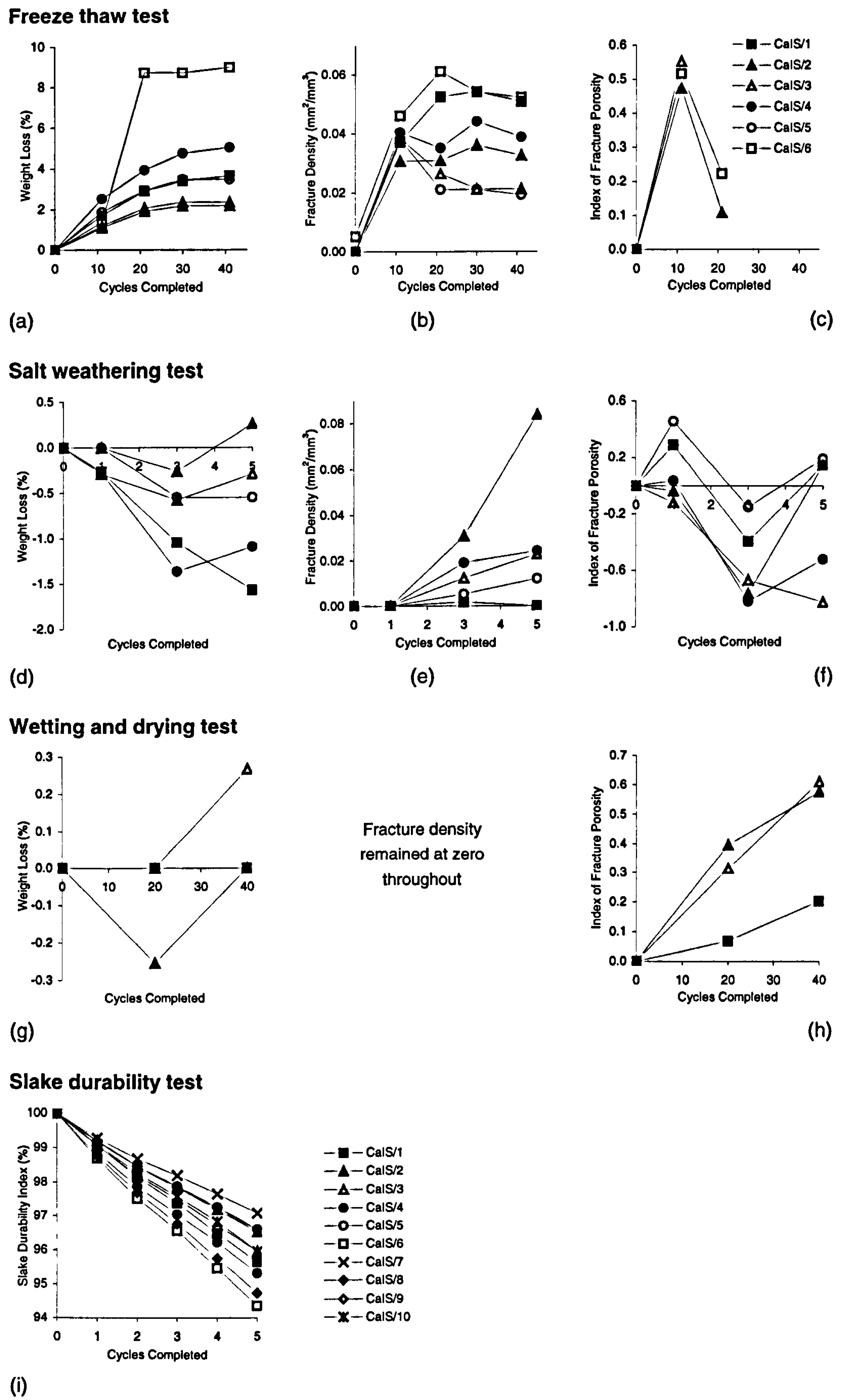


Figure 4.8 (a to i) Deterioration indicators for the calcareous sandstone **CalS**

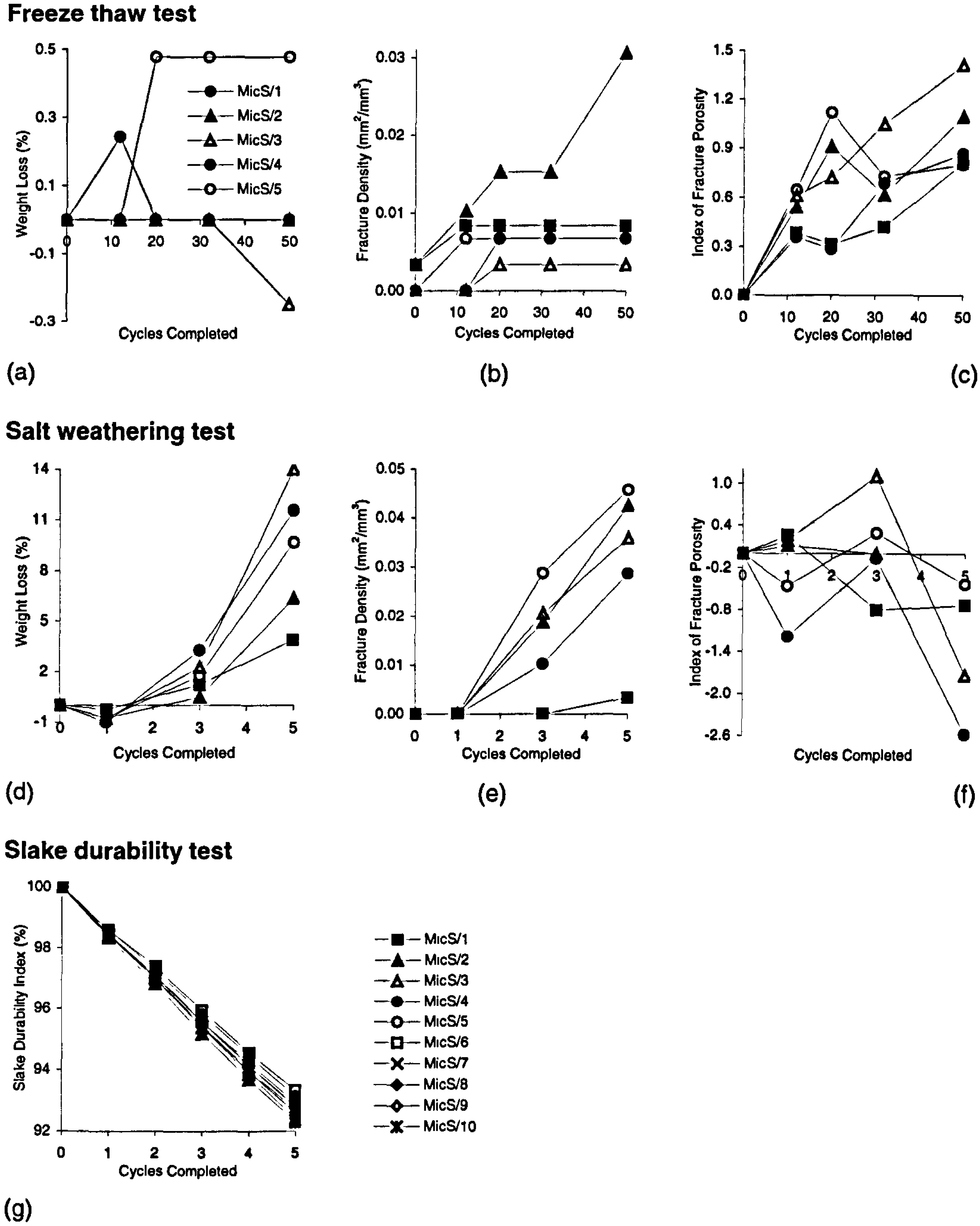


Figure 4.9 (a to g) Deterioration indicators for the micaceous sandstone **MicS**

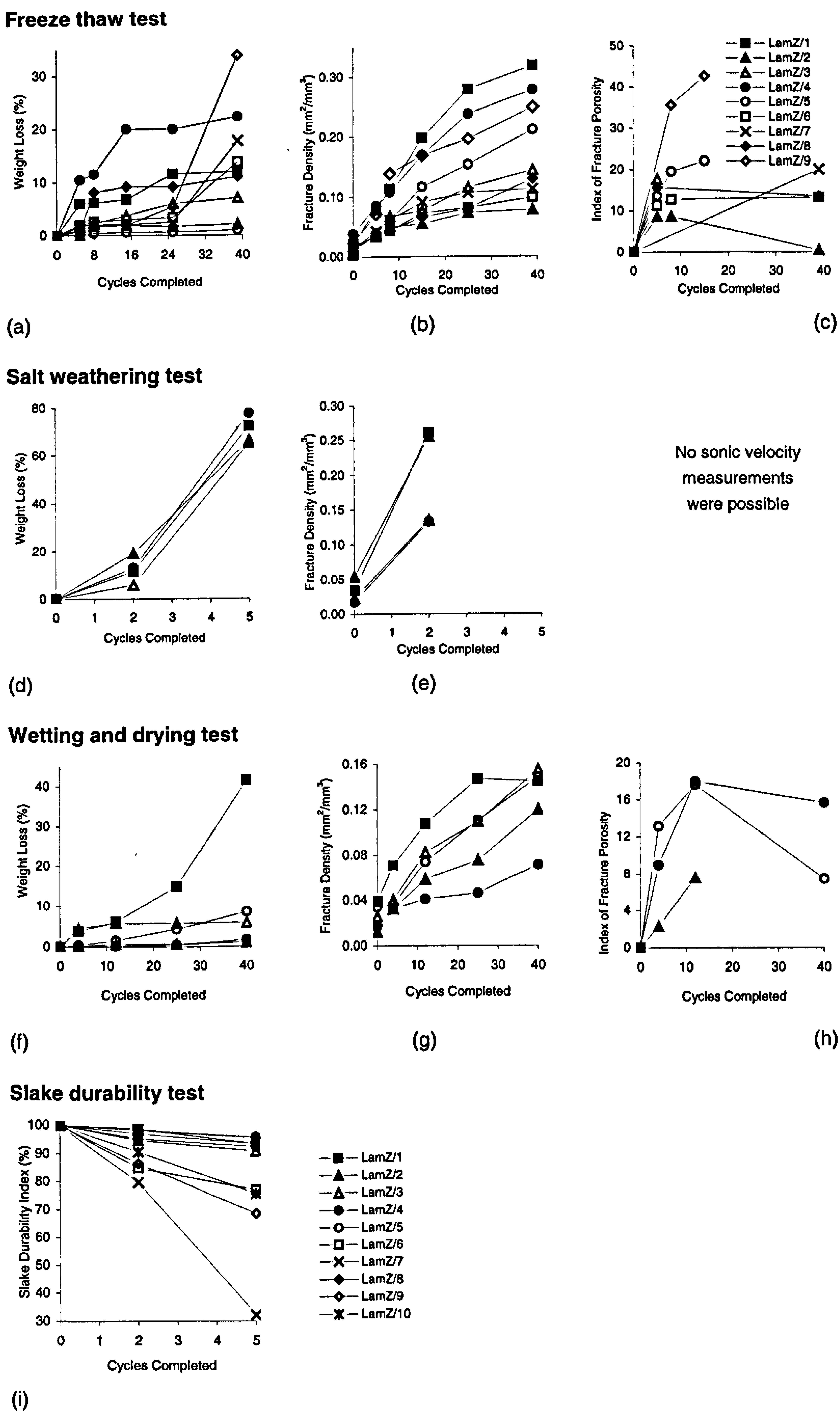


Figure 4.10 (a to i) Deterioration indicators for the laminated siltstone **LamZ**

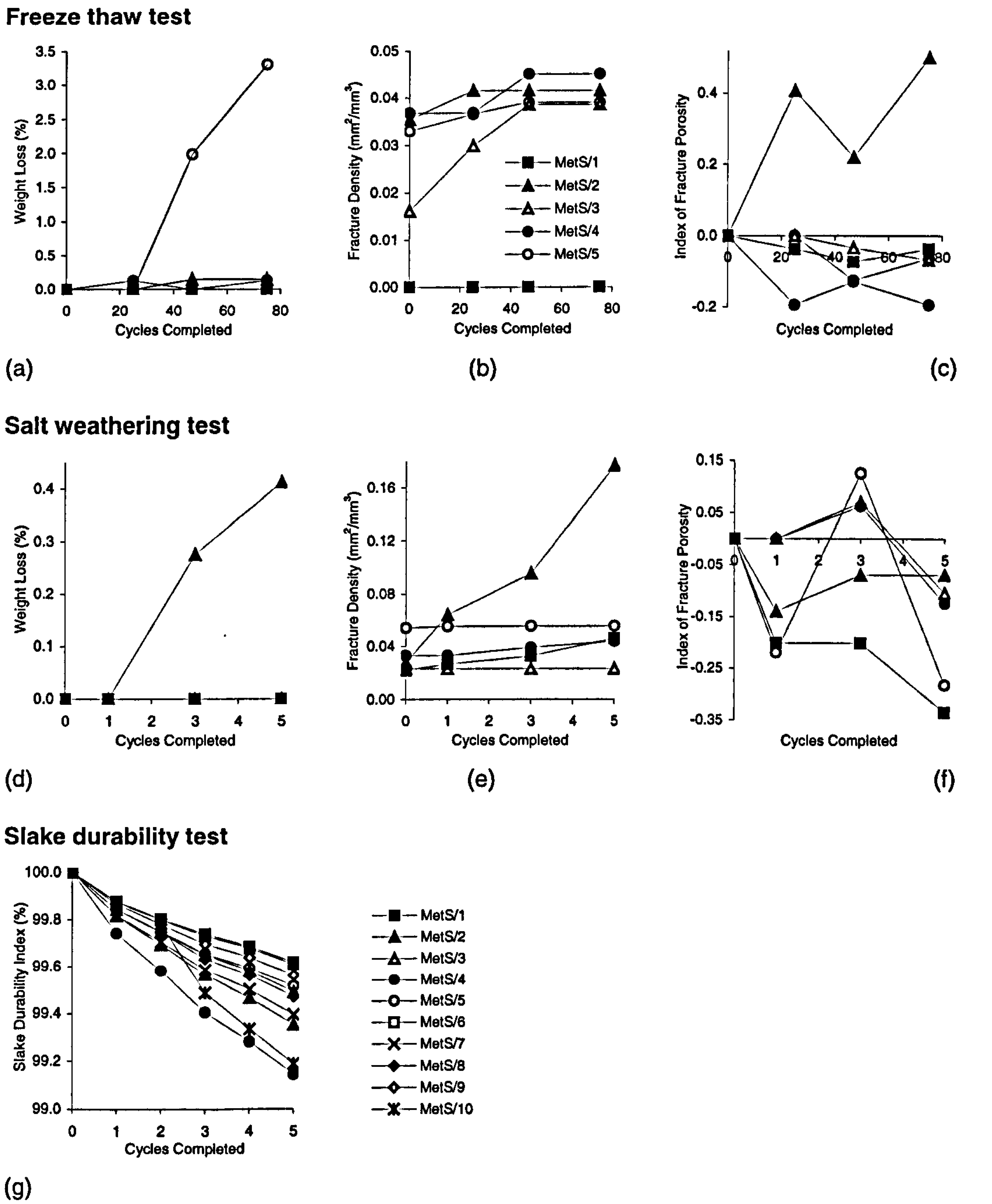


Figure 4.11 (a to g) Deterioration indicators for the metasediment **MetS**

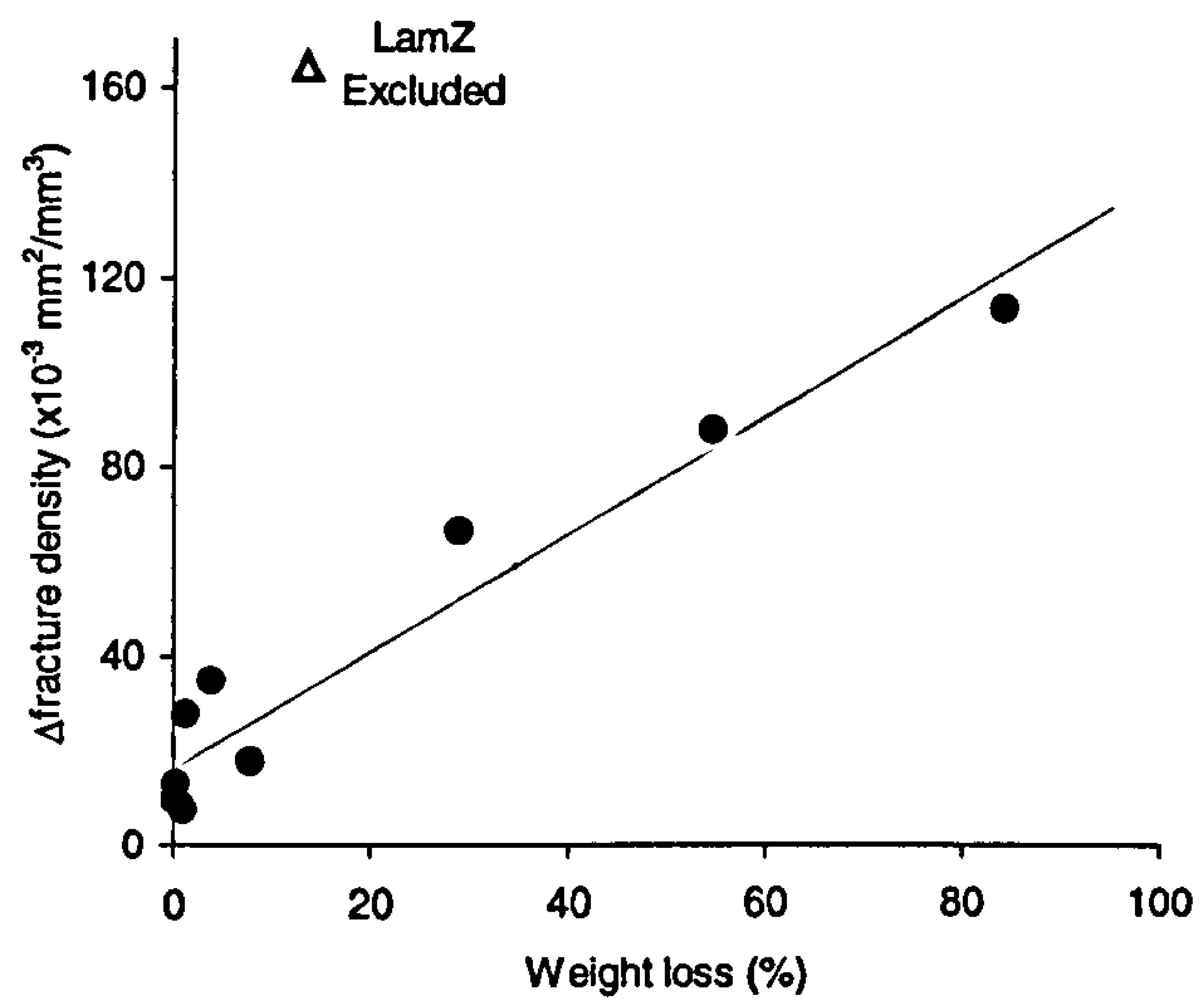


Figure 4.12 Correlation between Δfracture density and weight loss for the freeze-thaw test (r = 0.96)
(LamZ is shown in its correct position)

Freeze-thaw	Weight Loss	LRP Weight Loss*1	Change in Fracture Density	Final Fracture Density	Pre-Test Fracture Length	Final Fracture Length	Index of Fracture Porosity
LdCh (14)	60.74	74.33	87.9 **(6)**	92.5 **(6)**	39.17 **(6)**	17.28 **(6)**	n/a
MagL (50)	7.83	-	17.9	24.2	20.07	24.77	-0.05
OolL (21)*2	84.12	-	113.1 **(11)**	115.6 **(11)**	37.26 **(11)**	27.57 **(11)**	-
	-	-	<u>*28.0*</u>	<u>*28.0*</u>	<u>*25.14*</u>	<u>*26.32*</u>	<u>*2.35*</u> <u>*(5)*</u>
HdCh (39)	28.87	-	66.4	67.6	25.15	24.27	0.20
SpaL (75)	0.23	5.05	13.2	28.0	42.72	36.60	0.24
WeaS (50)	1.06	-	7.5	7.5	0.00	36.46	-0.41
CalS (41)	3.87	11.36	34.9	35.8	35.27	26.31	0.17 **(21)**
MicS (50)	0.05	-	9.8	11.2	29.03	29.21	1.00
LamZ (39)	13.5	-	164.42	180.4	29.08	28.79	11.84
MetS (75)	0.75	-	8.6	32.8	23.10	23.08	0.03

Table 4.1 Mean values of deterioration indicators for freeze-thaw*3
Where weight loss and LRP weight loss = %; fracture density = $\times10^{-3}$ mm^2/mm^3; fracture length = mm; fracture porosity = %.

Note 1 Weight loss based on the Largest Remaining Piece (LRP) only given where different from weight loss
Note 2 Underlined data relate to the single specimen OolL(2) only (refer to section 3.3)
Note 3 Bold figures in parenthesis refer to the number of cycles where less than the *total* number of cycles. Total number of cycles is given in parenthesis in column one.

The mean *change* in fracture density varied from 7.5x10^{-3} mm^2/mm^3 for WeaS (with a mean of 1.6 fractures per sample, of mean length 36mm) to 164x10^{-3} mm^2/mm^3 for LamZ (with a mean of 44 fractures per sample, of mean length 29mm). With the exception of MagL and LamZ, fracture density was greatest in the calcareous rocks. The two weakest rocks, LdCh (Figure 4.2b) and OolL (Figure 4.4b), and the highly laminated siltstone, LamZ (Figure 4.10b), suffered substantial fracturing and these rocks also had the greatest variation between individual specimens. Those rocks which resisted significant weight loss, SpaL (Figure 4.6b), WeaS (Figure 4.7b), MicS (Figure 4.9b) and MetS (Figure 4.11b), were also the most resistant to fracturing. All samples with the exception of HdCh (Figure 4.5b) showed more rapid development of fractures early on in the test, resulting in a convex curve of temporal variation.

Fracture porosity (Figure 4.1c): The mean index of fracture porosity varied from -0.41% (WeaS) to +11.84% (LamZ). Fracture porosity increased progressively throughout testing in one group, OolL (Figure 4.4c), MicS (Figure 4.9c) and LamZ (Figure 4.10c), though there was considerable variation between specimens. WeaS (Figure 4.7c) showed a small decrease in fracture porosity but this might be biased by one specimen which shows a large reduction. The remaining samples, MagL (Figure 4.3c), HdCh (Figure 4.5c), SpaL (Figure 4.6c), CalS (Figure 4.8c) and MetS (Figure 4.11c), showed inconsistent trends and considerable inter-specimen variation. Overall, stronger rocks produced lower index values and less inter-specimen variation, while weaker rocks acted conversely. Measurements were not possible for LdCh due to the severity of deterioration.

4.2.2 Salt weathering (Table 4.2)

Weight loss (Figure 4.1d): Mean weight loss varied from -4.6% (HdCh) to 70.3% (LamZ), with two samples, SpaL (Figure 4.6d) and MetS (Figure 4.11d), showing negligible change in weight. Many samples experienced an initial *gain* in weight due to the absorption of salt and its deposition in micropores. This is a phenomenon reported by others (eg Goudie 1974, Williams and Robinson 1998, T. Yates of Building Research Establishment, pers. com.) and can result in a temporary increase in rock strength (Williams and Robinson 1988) and consequent increased resistance to weathering (Booth 1990). Subsequently, and particularly after three test cycles, all of these samples with the exception of HdCh, either lost weight rapidly, or the initial gain in weight was reduced (OolL, CalS). This subsequent loss in weight presumably occurred either because (i) deposited salts were flushed out, facilitated by the enlargement of pore spaces due to deterioration, or (ii) particles and fragments were detached in response to salt weathering. Most commonly in LdCh (Figure 4.2c), MagL (Figure 4.3d), OolL (Figure 4.4d), WeaS (Figure 4.7d), CalS (Figure 4.8d) and MicS (Figure 4.9d), individual specimens within a sample followed a similar *trend* of weight loss but there was moderate variation in *rates* between specimens. HdCh (Figure 4.5d) and LamZ (Figure 4.10d) differed in showing very little variability between specimens. There was no obvious relationship between weight loss and rock type.

Fracture density (Figure 4.1e): An increase in fracture density occurred in all samples with a mean change varying from 15.1×10^{-3} (SpaL) to 164×10^{-3} (LamZ). Two distinct responses can be identified. In one group, LdCh (Figure 4.2d), MagL (Figure 4.3e), HdCh (Figure 4.5e) and LamZ (Figure 4.10e), fracture density increased substantially and the rate of deterioration between individual specimens was consistent. In the second group, OolL (Figure 4.4e), SpaL (Figure 4.6e), WeaS (Figure 4.7e), CalS (Figure 4.8e), MicS (Figure 4.9e) and MetS (Figure 4.11e), a smaller increase in fracture density took place, but there was also notably more variation between specimens. It is notable that the coarse, granular-textured rocks (ie the sandstones and the oolitic limestone) were among the most resistant to fracturing and the more homogeneous limestones were least resistant. In contrast to the freeze-thaw test, the temporal form of most fracture density curves was concave. This might be due to enhanced resistance early in the test as a result of the binding effect of absorbed salt. At the end of testing, WeaS was re-saturated in water and subsequently re-dried to see whether this would make any difference to the deterioration indicators. There was no change in weight loss, but fracture density increased from 15.7 to 37.8. This suggests that sufficient salt was retained in pore spaces to exert pressure due to crystallisation.

Salt weathering	Weight Loss	LRP Weight Loss*[1]	Change in Fracture Density	Final Fracture Density	Final Fracture Length	Pre-Test Fracture Length	Index of Fracture Porosity
LdCh	40.27	68.69	131.3 **(3)**	135.0 **(3)**	11.10 **(3)**	39.14 **(3)**	n/a
MagL	4.67	-	75.0	80.0	28.97	19.95	-6.40
OolL	-0.39	-	23.0	25.0	21.67	36.00	-1.07
HdCh	-4.57	14.28	115.5	117.6	18.48	34.59	4.75
SpaL	0.00	-	15.1	28.1	35.74	44.67	-0.25
WeaS	18.26	-	15.7	15.7	30.74	0.00	-5.73
CalS	-0.65	-	28.6	28.6	25.03	0.00	-0.17
MicS	9.09	18.34	31.2	31.2	28.33	0.00	-1.37
LamZ	70.30	87.49	164.2 **(2)**	196.1 **(2)**	34.71 **(2)**	28.78 **(2)**	n/a
MetS	0.08	9.38	37.0	69.1	27.18	33.66	-0.18

Table 4.2 Mean values of deterioration indicators for salt weathering*[2]
Where weight loss and LRP weight loss = %; fracture density = x10^{-3} mm^2/mm^3; fracture length = mm; fracture porosity = %.

Note 1 Weight loss based on the Largest Remaining Piece only given where different from weight loss

Note 2 Bold figures in parenthesis refer to the number of cycles where less than the *total* number of cycles (five in each case).

Fracture porosity (Figure 4.1f): The mean post-test fracture porosity varied from -6.4 (MagL) to +4.7 (HdCh), the latter being the only sample to achieve a positive mean value, indicating the addition of significant new void. This is possibly related to the intense incipient fracturing which took place. However, HdCh (Figure 4.5f) was also the only sample showing a gain in weight with no subsequent recovery (Figure 4.5d). This suggests that for this sample the amount of any new void introduced was sufficient to override the effect of pore infilling with salt. MetS (Figure 4.11f) showed very little change in fracture porosity, while some weaker rocks, MagL (Figure 4.3f) and WeaS (Figure 4.7f), produced high negative values and trends much more consistent than any found in the freeze-thaw. Other rocks, OolL (Figure 4.4f), SpaL (Figure 4.6f), CalS (Figure 4.8f) and MicS (Figure (4.9f), showed mainly negative values of smaller magnitude. Measurements were not possible for LdCh and LamZ due to the severity of deterioration.

4.2.3 Wetting and drying (Table 4.3)

Weight loss (Figure 4.1g): LamZ (Figure 4.10f) was the only sample to suffer a weight loss beyond the margin of error of the measurement method.

Fracture density (Figure 4.1h): LamZ (Figure 4.10g) was also the only sample where fracture density changed significantly, with a mean increase of 102.4. The temporal curve for these specimens was convex, in that rapid initial deterioration was followed by a tailing off.

Fracture porosity (Figure 4.1i): There was considerable intra-sample variation in fracture porosity for HdCh (Figure 4.5i), although the mean value was insignificant. Other samples showed a net increase in fracture porosity with a particularly high mean value of 11.6% being recorded for LamZ (Figure (4.10h). The low, but positive index values for LdCh (Figure 4.2g) and CalS (Figure 4.8h) might indicate internal pore modification due to wetting and drying, despite an absence of visible deterioration.

Wetting and drying	Weight Loss*[1]	Change in Fracture Density	Final Fracture Density	Pre-Test Fracture Length	Final Fracture Length	Index of Fracture Porosity
LdCh (80)	0.12	1.7	5.0	34.18	32.02	0.57
HdCh (39)	0.00	0.0	4.8	37.44	37.31	-0.07
CalS (40)	0.17	0.0	0.0	0.00	0.00	0.46
LamZ (40)	11.71	102.4	127.9	33.54	26.00	11.56

Table 4.3 Mean values of deterioration indicators for wetting and drying
Where weight loss = %; fracture density = $x10^{-3}$ mm^2/mm^3;
fracture length = mm; fracture porosity = %.

Note 1 Values for weight loss based on the Largest Remaining Piece were identical to weight loss

4.2.4 Slake durability (Table 4.4)

Slake durability	Slake Durability Index % (*range for individual specimens*)		Slake durability	Slake Durability Index % (*range for individual specimens*)	
LdCh	78.88	(75.03 ~ 84.25)	WeaS	93.49	(92.33 ~ 94.30)
MagL	95.99	(94.40 ~ 97.55)	CalS	95.90	(94.36 ~ 97.07)
OolL	95.37	(92.69 ~ 97.84)	MicS	92.75	(92.30 ~ 93.31)
HdCh	96.01	(94.46 ~ 97.04)	LamZ	89.71	(32.25 ~ 95.47)
SpaL	99.69	(99.64 ~ 99.74)	MetS	99.44	(99.15 ~ 99.62)

Table 4.4 Mean slake durability index values

The mean slake durability index (Figure 4.1j) varied from 99.7 (SpaL) to 78.9% (LdCh) and three distinct responses can be identified. For the two strongest rocks, SpaL (Figure 4.6g) and MetS (Figure 4.11g), an index in excess of 99% was recorded, while the weakest rocks, LdCh (Figure 4.2h) and LamZ (Figure 4,10i) (in terms of structural weakness) obtained the lowest values of 78.9 and 89.7% respectively. For the remaining rocks, values of between 93.5 and 96.0% were obtained, though there was a tendency for the calcareous rocks, MagL (Figure 4.3g), HdCh (Figure 4.5j) and CalS (Figure 4.8i), to be more durable than those with a granular texture, WeaS (Figure 4.7g) and MicS (Figure 4.9g). OolL (Figure 4.4g), with its granular oolitic texture, was perhaps exceptional in this respect. There was greater variability between individual specimens in the weaker rocks and also in OolL.

4.2.5 The relative susceptibility of samples to experimental weathering

A simplified method for making comparisons of weathering susceptibility between samples is to use a ranking process. This is particularly useful given that there are not only considerable variations within samples but also between the different indicators of deterioration. The method also effectively normalises data so that comparison across different deterioration indicators can be made.

4.2.5.1 Ranking procedure

Samples were ranked in descending order of susceptibility (Table 4.5), so that the least susceptible sample was ranked 10, and the most susceptible ranked 1. Where there were fewer than 10 samples (eg for fracture porosity following freeze-thaw and salt weathering), the ranking order was begun at 10 (so ranks 1 and 2 are missing). For wetting and drying where only four samples were tested, rank numbers 1, 4, 7 and 10 were used to ensure some comparability with other test rankings. Negative values of weight loss and fracture porosity were given higher rankings (indicating lower susceptibility). This is not an entirely satisfactory way of dealing with negative data. Where negative weight loss occurs due to salt deposition, for instance, the ranking makes the probably false assumption that this condition equates with greater durability. Alternately, the ranking also assumes that a reduction in fracture porosity contributes more to rock durability than *no change* in fracture porosity. An alternate way of dealing with negative values is to rank them on the basis of a visual estimate of breakdown (eg Goudie 1999). However, this is also unsatisfactory because it involves ranking qualitative information. In Table 4.5, rankings for negative values are given in parenthesis to facilitate interpretation.

4.2.5.2 Sample rankings

For the freeze-thaw test, there is good correlation in ranking between weight loss and fracture density and also for fracture porosity, with the exception of SpaL and MicS. This has several implications: (i) It suggests that for this test, a crude measure of durability could be obtained by using *either* weight loss *or* fracture density as the sole indicator. (ii) MicS and SpaL both showed a much higher ranking for fracture porosity than would have been expected on the basis of weight loss and fracture density results, suggesting that 'hidden' or internal modification might have taken place. In fact, all individual specimens for MicS (Figure 4.9c) showed a distinctive, progressive increase in fracture porosity from the start of testing. In theory, this type of trend could have been caused by the gradual opening of a single crack but this is highly unlikely to have been the case in all five specimens. It is much more likely that the increase represents progressive microcracking or pore modification. This might be a pre-cursor to more visible modification had experimental frost weathering of longer duration or greater intensity been conducted. (iii) The close accord between the three deterioration indicators suggests that the three processes of which they are a reflection, ie detachment (weight loss), fracturing (fracture density), microcracking and pore modification (fracture porosity), operate in parallel and are equally affected by frost weathering.

	LdCh	MagL	OolL	HdCh	SpaL	WeaS	CalS	MicS	LamZ	MetS
Freeze-thaw										
Weight loss	2	5	1	3	9	7	6	10	4	8
Fracture density	3	6	2	4	7	10	5	8	1	9
Fracture porosity	n/a	(9)	3	6	5	(10)	7	4	2	8
Mean	**2.5**	**6.7**	**2.0**	**4.3**	**7.0**	**9.0**	**6.0**	**7.3**	**2.3**	**8.3**
Standard deviation	0.7	2.1	1.0	1.5	2.0	1.7	1.0	3.1	1.5	0.6
Salt weathering										
Weight loss	2	5	(8)	(10)	7	3	(9)	4	1	6
Fracture density	2	4	8	3	10	9	7	6	1	5
Fracture porosity	n/a	(10)	(7)	3	(6)	(9)	(4)	(8)	n/a	(5)
Mean	**2.0**	**6.3**	**7.7**	**5.3**	**7.7**	**7.0**	**6.7**	**6.0**	**1.0**	**5.3**
Standard deviation	0.0	3.2	0.6	4.0	2.1	3.5	2.5	2.0	0.0	0.6
Wetting and drying										
Weight loss	7			10			4		1	
Fracture density	4			7			10		1	
Fracture porosity	4			(10)			7		1	
Mean	**5.0**			**9.0**			**7.0**		**1.0**	
Standard deviation	1.7			1.7			3.0		0.0	
Slake durability										
Inverse of weight loss	**1**	**7**	**5**	**8**	**10**	**4**	**6**	**3**	**2**	**9**

Table 4.5 Ranking of samples according to deterioration indicator

For the salt weathering test, there is generally good correlation in ranking for weight loss and fracture density and fracture porosity also correlates moderately well with fracture density. However, there is poor accord between weight loss and fracture porosity probably because of the influence of salt deposition in pore spaces. The measurement of fracture porosity is likely to be much more sensitive to pore infilling than change in weight loss, simply because of the increased sensitivity of the equipment used.

Exceptionally, HdCh and WeaS did not show a good correlation between weight loss and fracture density, but both did show a perfect correlation between fracture density and fracture porosity. HdCh was distinct in its response to salt weathering in that, although it developed a dense network of incipient fractures, specimens generally remained intact. Thus, although fracture density was particularly high, weight loss remained the lowest of all samples for this test. It would appear that external fracturing was closely matched by internal modification as indicated by fracture porosity. It is possible that the binding effect of salt could partly explain why specimens remained intact, but this is unlikely to account for all of this effect. In contrast, results for WeaS indicate a lack of fracturing but a relatively high weight loss. It is likely that due to its coarse granular texture, material loss from specimen surfaces was able to occur without any corresponding structural changes. This suggests general resistance to weathering at the mass scale and weakness at the material scale. A similar pattern of deterioration, though less severe, was also indicated for the freeze-thaw test.

There was very good correlation between all deterioration indicators for the wetting and drying test and any variations which do occur simply reflect the fact that values recorded were extremely low. LamZ was the exception, being much more susceptible to deterioration than the other samples.

4.2.6 The relative severity of weathering tests

A similar ranking process can be used to analyse the relative effect of the different weathering tests on each rock type.

4.2.6.1 Weathering test rankings

Sample	Weight loss	Fracture density	Fracture porosity
LdCh	F > M > S >>> W	M >> F >>>> W	*3
MagL	F > M > S	M >>> F	(F) > (M)
OolL	F >>>> S > (M)	F >>>> M	F > (M)
HdCh	F >> S > W > (M)	M >> F > W	M > F > (W)
SpaL	S > F > M	M > F	F > (M)
WeaS	M > S > F	M > F	(F) > (M)
CalS	S > F > W > (M)	F > M >> W	W > F > (M)
MicS	M > S > F	M >> F	F > (M)
LamZ	M >>> F > W > S	F = M >> W	F > W *3
MetS	F > S > M	M >> F	F > (M)

Table 4.6 Ranking of weathering test for each sample*1,2
Where F = freeze-thaw; M = salt weathering (magnesium sulphate)
S = slaking (weight loss only); W = wetting and drying.

Note 1 The symbol '>' is used in a relative sense only.
Note 2 Notation in parenthesis indicates negative values
Note 3 Measurement of fracture porosity was not possible for LdCh (F and M) and LamZ (M only).

In Table 4.6, the ranking of weathering tests is given for each rock, with respect to the three deterioration indicators. Among the calcareous rocks (LdCh, MagL, OolL, HdCh) there is a clear suggestion that freeze-thaw is more likely to result in weight loss than it is in the sandstones (WeaS, MicS, LamZ). In these, weight loss is more likely to be associated with salt weathering. These results seem to reflect fundamental differences in the mode of deterioration for the two different sets of rocks. The greater weight loss observed in the limestones occurred after freeze-thaw as a result of large fragments of rock being broken away along fractures. In contrast, most of the sandstones, with the exception of LamZ, remained intact. In the sandstones after salt weathering, most weight loss occurred as a result of granular disintegration, surface pitting and minor fragmentation, whereas with the exception of LdCh, the limestones were much more likely to remain intact. The limestones are generally much finer grained (excepting OolL) with a relatively high microporosity, compared to the generally coarse grained sandstones (excepting LamZ) with a low microporosity (also excepting LamZ). This suggests that the finer grained, high microporosity rocks are more susceptible to freeze-thaw, and that the coarse grained, low microporosity (and large modal pore diameter) rocks are more susceptible to salt weathering. Relationships between susceptibility and rock properties are considered further in section 4.6.

It is notable that a greater density of fractures is more likely to result from salt weathering than freeze-thaw, the only exceptions to this being OolL and CalS. OolL was remarkably resistant to salt weathering given its severe response to freeze-thaw. The rankings for fracture porosity simply confirm that positive values or small negative values occur mostly in association with freeze-thaw, whereas negative values occur due to salt weathering from deposition of salt in pores. The exception to this is HdCh which was discussed in sections 4.2.2 and 4.2.5.2 above.

The rankings in Table 4.6 highlight the fact that for all deterioration indicators, wetting and drying and slaking are likely to result in least rock deterioration. Even in LamZ which was highly susceptible to all tests, wetting and drying and slaking ranked third and fourth behind freeze-thaw and salt weathering for weight loss. It would appear that slaking has a more significant effect on sandstones (WeaS, CalS, MicS) than other rocks, a reflection of the interaction between their granular texture and the abrasive nature of the test. The rankings appear to suggest that slaking is important in the breakdown of SpaL and MetS, the two strongest rocks. In fact, these were extremely resistant to weathering generally and also the ranking procedure tends to exaggerate small values.

4.2.7 Discussion

4.2.7.1 The role of weathering processes in severity of rock deterioration

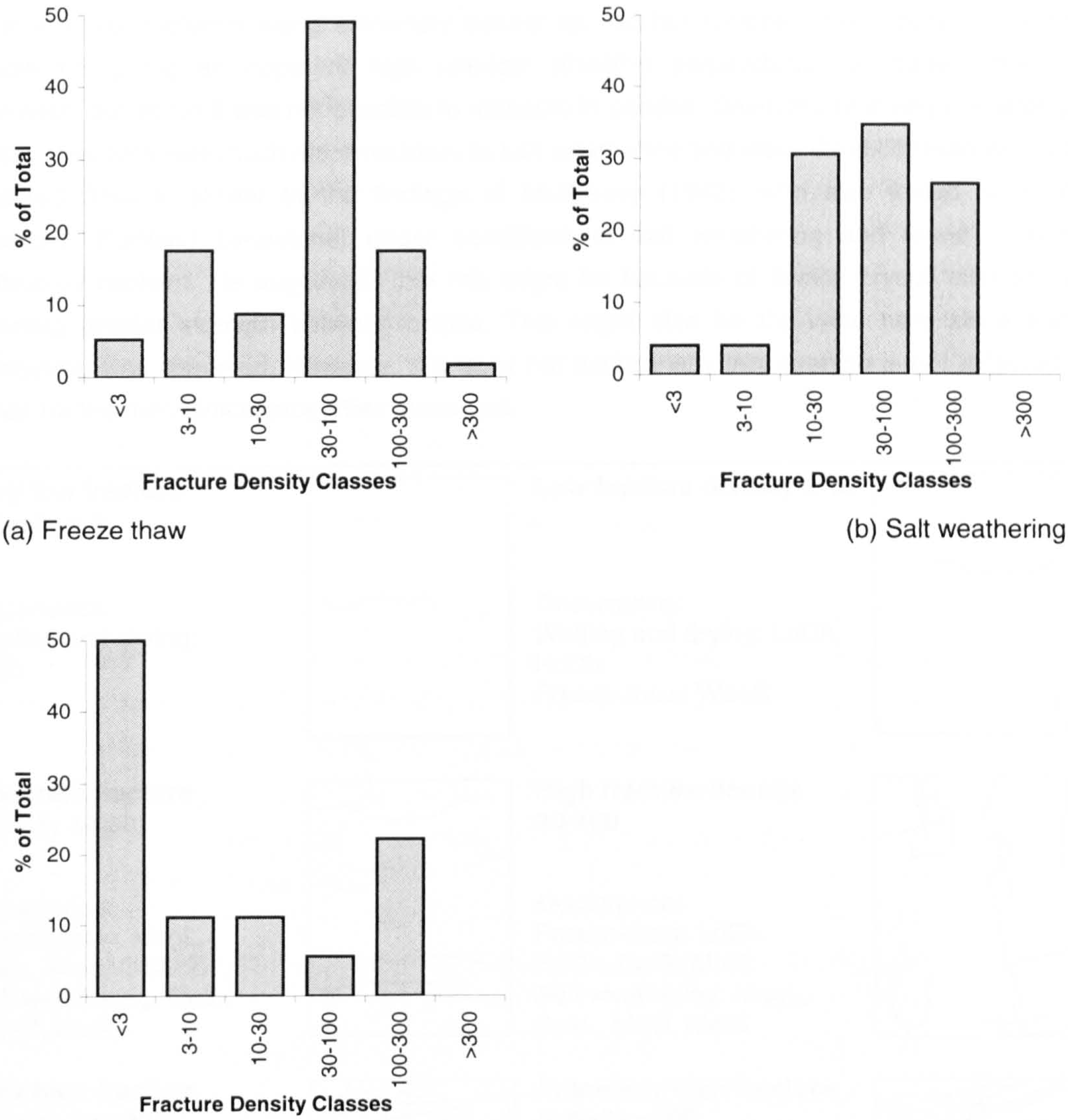
In terms of their response to each weathering test, the rocks can be broadly divided into four groups. The first group represents rocks which had a high resistance to weathering, and comprises SpaL and MetS. The second group represents rocks which were a little susceptible to all of the weathering tests, and comprises MagL and CalS. The third group represents rocks which had a very low resistance to weathering, and comprises LdCh and LamZ. The fourth group represents rocks which had a variable response to weathering, and comprises OolL, HdCh, WeaS and MicS. OolL, for example, was severely affected by the freeze-thaw test but only minor deterioration occurred due to salt weathering and slake durability. HdCh also suffered moderate deterioration due to freeze-thaw and salt weathering, but was much more resistant to wetting and drying and slake durability. The two sandstones, WeaS and MicS, showed a similar pattern, in being very resistant to freeze-thaw, but susceptible to salt weathering and slake durability.

These results indicate that in some cases rock properties such as rock strength or structure, determine the response to weathering, which is almost always similar, regardless of the environmental conditions. In addition to mechanical and structural properties, lithological properties such as mineral composition might also be important. The absence of clays and other platey minerals in LdCh, for instance, could explain its complete resistance to wetting and drying despite its very high susceptibility to other tests. In other cases, the environmental regime to which the rock is subjected appears to be the controlling influence on deterioration response, although clearly it is the interaction between the weathering process and rock properties which determines this response in each case. Nevertheless, it is also clear from the results that some weathering tests are generally more severe than others. In terms of weight loss, for instance, freeze-thaw generally results in more severe deterioration than salt weathering, while wetting and drying and slake durability do not generally cause significant weight loss. Exceptions to this are LamZ, where weight loss due to salt weathering was more than five times that for freeze-thaw, and LdCh, where slake durability caused a weight loss of more than 20%. WeaS and MicS also suffered greater weight loss due to salt weathering than from freeze-thaw. Considerable caution must therefore be exercised when attempting to correlate deterioration susceptibility with particular environmental conditions or weathering processes. Nevertheless, the clear implication of the results is that rock durability and the rock properties which influence it should not be considered in isolation, but with specific reference to the environmental conditions at issue.

4.2.7.2 The use of different deterioration indicators

The results here show sufficient variation between deterioration indicators to suggest that while a sole indicator can provide a crude measure of durability, this is unlikely to provide a full picture of the mechanisms of deterioration taking place. In particular, if fracture density and weight loss are both high, deterioration will probably be primarily by large scale physical breakdown. In contrast, a high weight loss and low fracture density might indicate granular breakdown of the type observed in the sandstones. If both fracture density and fracture porosity are high, this might indicate internal or superficial damage which does not result in physical loss of material, or in some cases, visible deterioration. An ideal solution for rock weathering studies would be to utilise a range of deterioration indicators. Aside from those used here, percentage change in elasticity could also be used (eg Allison 1988, 1990), or pre- and post-test comparison using destructive methods such as the micropetrographic index (Irfan and Dearman 1978).

Given the usefulness of the measurement fracture density, it was decided that it would be helpful to devise a simple classification for future use. A five-fold classification was originally devised, with divisions <20; 20-60; 60-120; 120-160; >160 ($x10^{-3}$ mm^2/mm^3), but this produced a skewed distribution for the data collected here and caused considerable 'bunching' of data at the lower end of the scale. Accordingly, it was decided to test the applicability of the 'fracture intensity' classes devised by Fookes and Denness (1969) in their field based classification. Their classification was based on in situ excavation, examination and measurement of fissure surfaces in Cretaceous sediments and was designed to characterise fissures at the mass scale using units m^2/m^3. In fact, if these units are transposed to mm^2/mm^3, the values obtained fit the data presented here remarkably well, suggesting that there is repetition of fracture patterns at mass and material scales. Using these classes, frequency histograms showing fracture densities for each test are given in Figure 4.13 (a-c), and those for freeze-thaw and salt weathering show reasonably 'normal' distributions. The classification, slightly modified from the Fookes and Denness (1969) original, is given in Figure 4.14 together with illustrations of specimens representative of each class.



(a) Freeze thaw

(b) Salt weathering

(c) Wetting and drying

Figure 4.13 (a-c) Frequency histograms for fracture density classes

4.2.7.3 Sample deterioration in relation to intrinsic rock strength

Unsurprisingly, the combined results of weight loss, fracture density and fracture porosity for each of the tests conducted indicate that the weakest rocks are also the least resistant to weathering in general. Conversely, the strongest rocks are most resistant to weathering. Such a result is not unexpected since compressive strength relates closely not only to tensile strength, but also to properties which influence porosity such as texture and packing. The role of both tensile strength and porosity has been acknowledged in frost weathering (Matsuoka 1990b). While most of the remaining rocks occupy the middle ground in terms of compressive strength and durability, there are three notable anomalies. Both MagL and WeaS are more durable under all test conditions than their basic mechanical strength suggests they should be. Consideration is given in the following sections of this chapter to mechanical and lithological properties which might explain this anomaly. At the opposite end of the extreme is the laminated siltstone (LamZ) which is the least durable of all samples, yet has a similar compressive strength to SpaL. The obvious explanation for this anomaly lies in the structure of LamZ, which led to intense fracturing along extremely closely spaced laminations. This rock is also highly anisotropic, giving an apparent high strength (81MPa) perpendicular to these planes of weakness, but which it was not possible to measure in parallel. Given the relatively low strength of OolL, this rock was much more resistant to salt weathering and slake durability than would be expected. This is similar to the findings of McGreevy (1982), who also tested an oolitic limestone (Portland Limestone) under conditions of salt weathering and found it to be particularly resilient. He suggested that this might be because of limited crystal interlocking, imparting greater strength between ooliths. This might also be the case here since some interlocking was observed. However, this does not explain why this strength is not reflected in values for the mechanical properties measured.

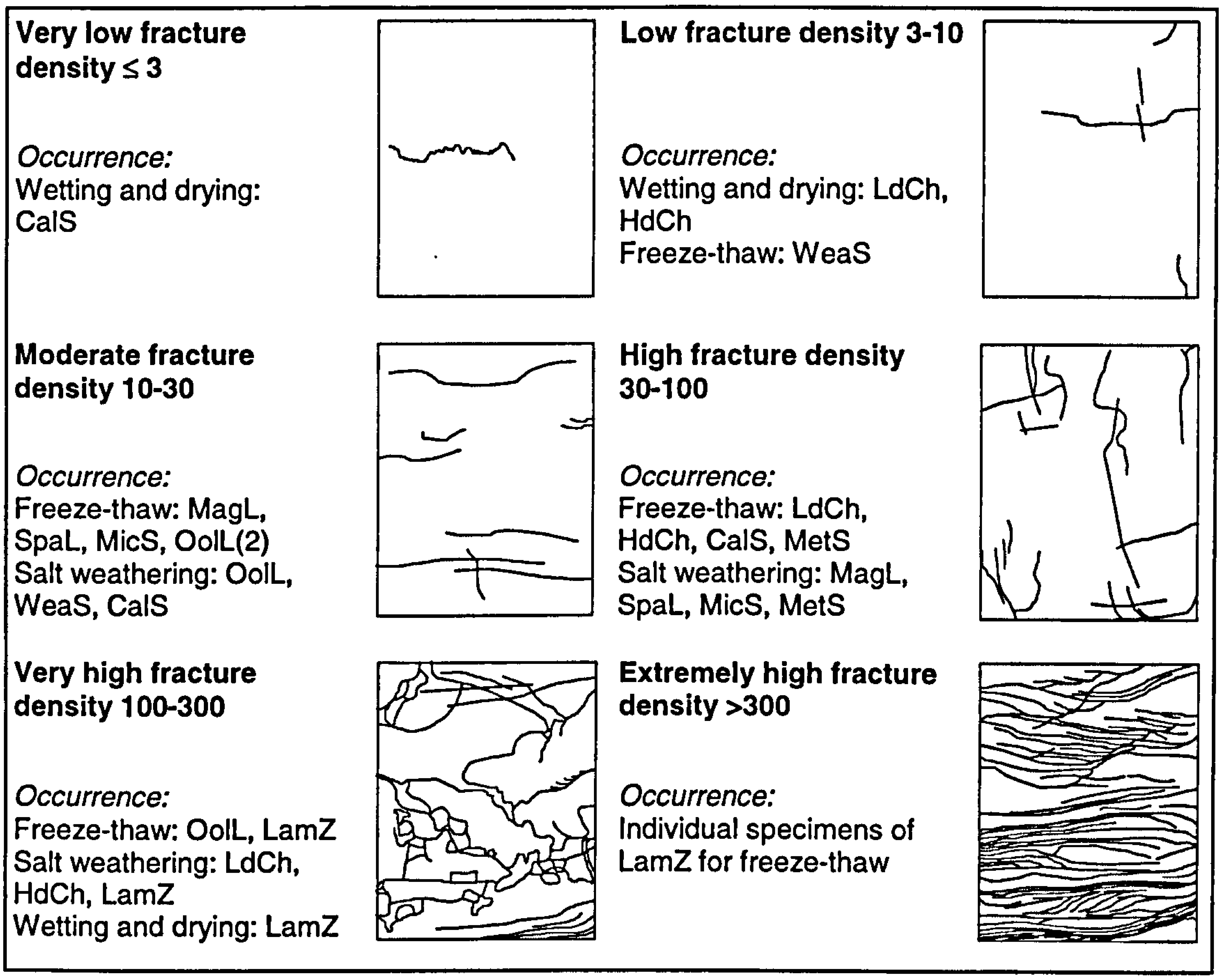


Figure 4.14 Classification of fracture density (units are $x10^{-3}$ mm^2/mm^3)
Sketches are indicative fracture densities based on the specimens tested

4.3 Results of Rock Property Measurements

In the second part of this chapter, the focus is on some of those properties generally regarded as exerting the greatest influence on susceptibility of rock materials to weathering processes. These include mechanical strength (Cooks 1983), elasticity (Cooks 1983; Allison 1988, 1990), pore size distribution (Winslow and Lovell 1981; Ordonez et al 1997), effective porosity (McGreevy 1982, 1996) and saturation coefficient (Everett 1961, Richardson 1991). Definitions of these properties and the factors affecting them were considered in section 2.3 of Chapter

Two and will not be repeated here. First, the property values for the ten rock samples investigated are presented and discussed. Then, correlation between these properties and deterioration severity is explored. The primary aim is to evaluate the causes of deterioration with specific reference to some void-dependent and mechanical properties.

4.4 Pore Dependent Properties

Data for all pore dependent properties are given in Table 4.7 on page 94.

4.4.1 Dry density (ρ)

In most cases, density values for the rocks are considerably less than that of the mineral constituents, reflecting the presence of pore spaces within. For example, the low density chalk with a ρ of 1.74g/cm^3 and n_e of 33% is thought to have been partially cemented at grain contacts prior to burial by overburden (Bell et al 1990). Thus the reduction in pore space due to consolidation was much less than that which occurred in the uncemented Yorkshire Chalk (HdCh) (Clayton 1983). In other cases, the ρ value approaches the specific gravity of the dominant minerals contained in the rock. This is true for SpaL with a ρ of 2.66g/cm^3, dominated by calcite with a specific gravity of 2.71 (Read 1956), and also for MetS with a ρ of 2.65g/cm^3, dominated by quartz with a specific gravity of 2.65 (Read 1956). The lowest ρ was recorded for the Magnesian Limestone and aside from the high ρ rocks already mentioned, most other values lay between 1.9 and 2.2g/cm^3.

4.4.2 Water absorption capacity (W_{ab}), effective porosity (n_e) and total porosity (n_t)

Since one of the primary determinants of density is the volume of pore space, it is to be expected that there should be a good correlation between ρ and n_e or W_{ab}. Reference to the data in Table 4.7 shows this generally to be the case. The values for n_e range from 0.5 to 33%, the lowest being for the sparry limestone, metasediment and laminated siltstone and the highest being for the two chalks. Both the n_e and n_t values highlight variations between the rock types and the difference between these properties is explored further in the next section.

4.4.3 Saturation coefficient (S)

For each rock type S was obtained twice, once using slake cube sized specimens and the other using the main test cylinders. The results, given in Table 4.7, show mean values ranging from 0.54 to 1.12. The differences between the two sets of data might indicate a scale effect in the measurement method. If this is the case, S_2 values are more strictly comparable with ρ, W_{ab}, n_e and n_t data, which were also obtained from main test cylinders. S_1 values might be more comparable to data obtained from mercury intrusion porosimetry where small test cubes were used. Mean values, S_3, have been used in the analyses presented in section 4.6. Strictly, values in excess of 1.0 should not be possible but it is notable that it was the three very low porosity rocks for which such values were measured. Tourenq (1970) also observed the tendency for particularly low porosity rocks to yield apparently high saturation coefficients. It is likely that because of the low porosity values involved, minor inaccuracies in measurements would be grossly magnified. It is also highly likely that surface water (or even water locked in part-open fractures in the case of LamZ and MetS) could account for the apparently excessive moisture contents of these rocks.

It could be expected that rocks with a low S might contain a large proportion of fine pores which water could not easily access. The findings of Honeyborne and Harris (1958) and McGreevy (1982) suggest that in fact the opposite is true, that low S occurs in rocks with a high proportion of coarse pores. The suggestion is that the suction effect of fine pores might increase the extent to which water was drawn into the rock, rather than limiting access. It is unclear at this stage whether the low S recorded for the magnesian limestone reflects (i) the high proportion of coarse pores, or (ii) the large cavities present at the specimen surface which might have reduced the M_s value measured. This is discussed further in section 4.4.5.

4.4.4 Porosity as determined from mercury intrusion porosimetry (n_m)

The values for n_m range from 0.4 to 39%, the general ranking of values being as for n_e. However, there are some notable differences between porosity obtained using the two different methods (ie free saturation and mercury intrusion). These differences reflect a fundamental contrast in the fluid used for measurement and the fact that in the mercury intrusion technique, high pressure is applied. Porosity measured using mercury impregnation (n_m) usually gives a value which is greater than n_e (eg as for LdCh, MagL, HdCh, SpaL, WeaS and CalS) but less than *total* porosity (n_t) (eg as for MagL, OolL, HdCh, WeaS, MicS, LamZ and MetS). Cases where n_m is less than n_e might be due to minor sample variability because it is not possible to conduct pre- and post-test mercury intrusion measurements on the same test specimen. This might apply to MicS, LamZ and MetS, though differences between n_e and n_m for the latter is more likely to relate to measurement error because of the low values involved. However, two more significant anomalies can be identified which are difficult to explain. The first concerns MagL where n_m was more than double the n_e value (considered further in section 4.4.5), and the second concerns OolL where n_m was less than half the n_e value. The n_m value for the oolitic limestone might have been made on an unrepresentative specimen, despite strenuous efforts to avoid this. This is indicated by the fact that post-freeze-thaw and salt weathering n_m values (18.34% and 15.47% respectively) were much more comparable to the pre-test n_e value of 17.12% than the pre-test n_m of 7.62%.

In most of the cases where n_t was less than n_m, the difference is small and can probably be explained by specimen variation. Since values of n_e represent mean sample values, with pre- and post-test measurements being made on the same piece of rock, they might be more reliable for pre- and post-test comparisons than mercury intrusion data.

4.4.5 Pore size distribution and microporosity (μn_m)

Pore size distributions are depicted in Figure 4.15 as cumulative percentage mercury intrusion volume. The same data are also presented as cumulative pore volume in Figure 4.16 because, while the former clearly illustrates the distribution of pore sizes, the latter is more useful for comparison of porosity between rock types. The extent to which these pore size distributions

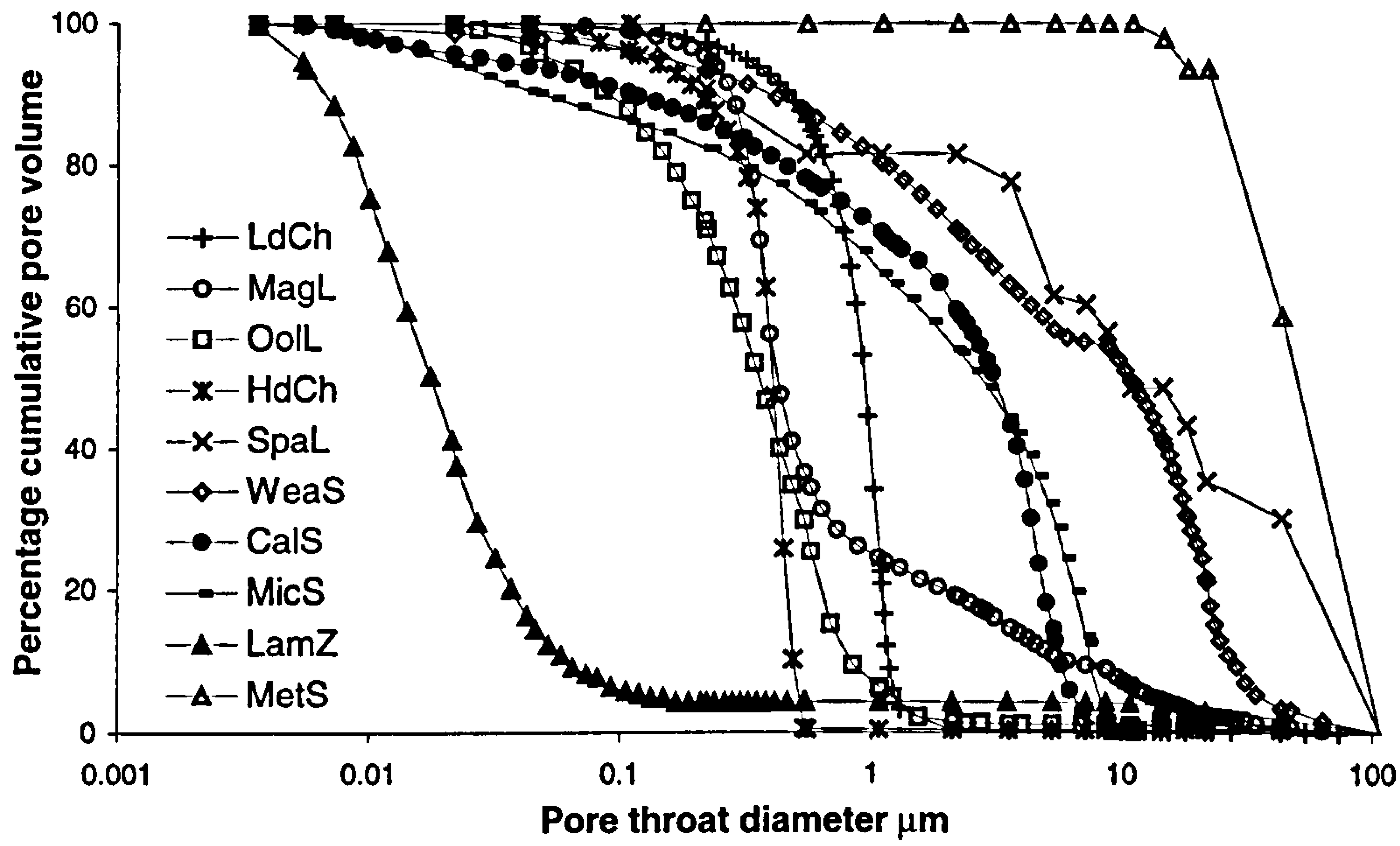


Figure 4.15 Pre-test percentage cumulative pore volume

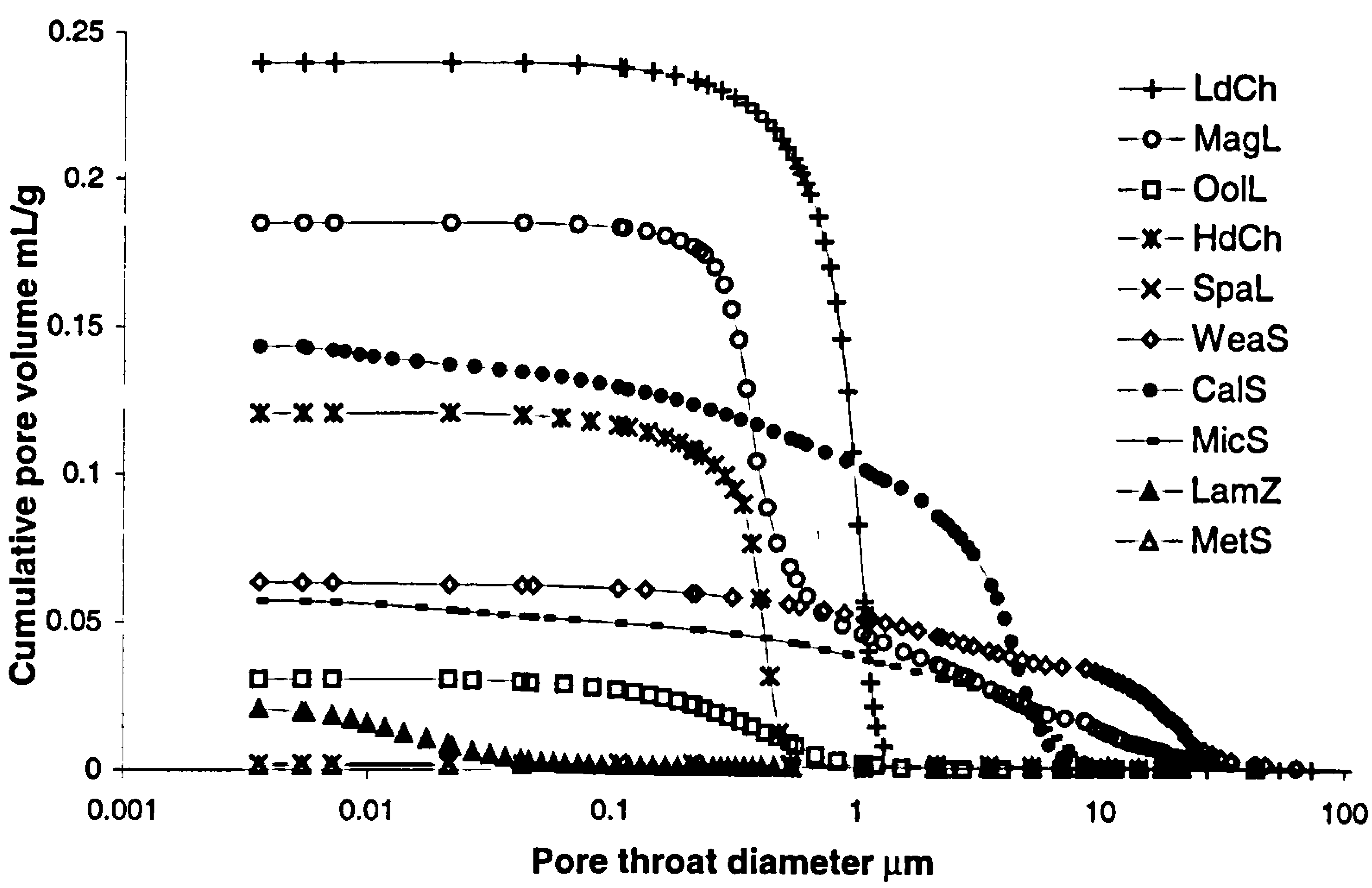


Figure 4.16 Pre-test cumulative pore volume mL/g

reflect actual pore sizes is tested in the scanning electron microscope analyses described in section 4.4.6. Pore size distributions for the sparry limestone and metasediment are unreliable because of their extremely low porosity, which was below the resolution of the mercury impregnation technique.

Microporosity is defined here as the percentage of pores less than 1μm (after McGreevy 1982, 1996). Most of the rock types tested possess micropores, with the possible exception of MetS. The percentage of micropores was very low for SpaL with the three sandstones (WeaS, CalS, MicS), varying from 18 to 35%. Microporosity for OolL, HdCh and LamZ was extremely high, in all cases exceeding 93%, with the chalk containing less than 0.5% macropores. However, similarity of microporosity between rocks does not mean similarity of pore size distribution as comparison of these latter three rocks shows. The remaining rocks, LdCh and MagL had moderately high microporosity of 65 and 75% respectively.

The laminated siltstone stands apart from other rocks in containing a particularly high percentage of extremely fine pores, with 94% being less than 0.1μm. The two chalks differed from other rocks in that their pore sizes lay within extremely narrow limits, with 50% of pores lying between 0.76 and 1.09μm for LdCh and 50% pores lying between 0.34 and 0.45μm for HdCh. In contrast to this, the sandstones had a particularly wide distribution of pore sizes. It is notable that WeaS also differed from other rocks in having a massive 53% of pores in excess of 10μm. The similarity of pore size distribution and microporosity between CalS and MicS suggests that they might show similarity in their durability to different weathering conditions.

The relationship between saturation coefficient and microporosity was briefly discussed in section 4.4.3. Data for the rocks tested here are shown in Figure 4.17 and can be interpreted in two different ways. (i) If the data values for SpaL and MetS are taken as an accurate reflection of their moisture absorption capacity, and not just anomalous data due to their very low porosity, then there would appear to be a negative correlation between S and μn_m (Figure 17a). In other words, with increasing percentage of fine pores, S decreases. In this scenario, two trends are evident. One trend includes MagL and the sandstones, in which variations in μn_m have a large impact on S. The other trend includes the chalks, OolL and LamZ where large reductions in μn_m produce relatively little increases in S. (ii) An alternate interpretation is to accept that SpaL and MetS values of S and μn_m are unreliable. In this case (Figure 4.17b), a clear positive correlation between μn_m and S emerges, with MagL present as an anomalous point. The correlation coefficient for the remaining seven rocks is 0.91. This interpretation supports the findings of Honeyborne and Harris (1958) and McGreevy (1982), that a higher proportion of fine pores increases absorption of water into the rock.

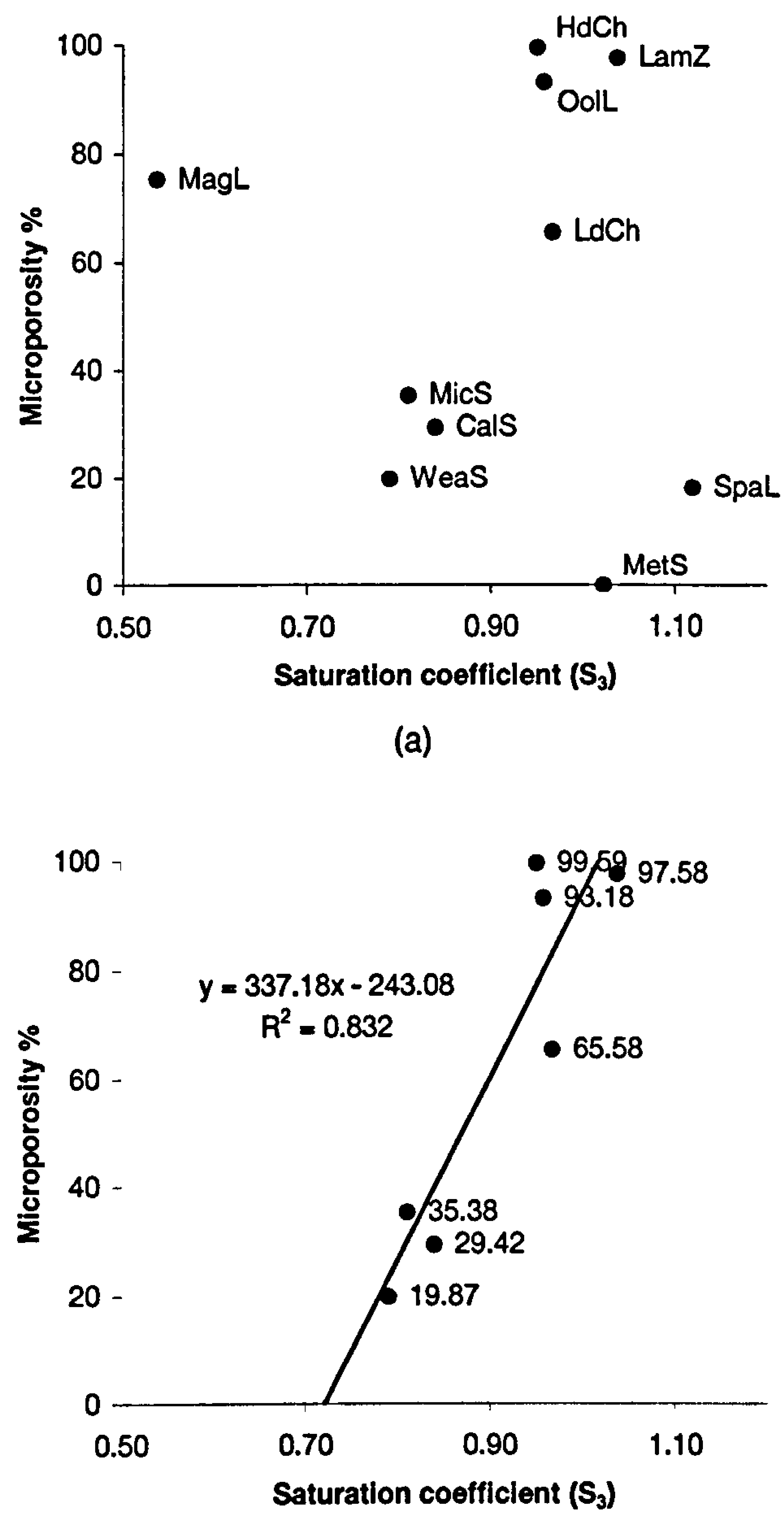


Figure 4.17 (a and b) Trends for microporosity and saturation coefficient (based on S_3)
Points for MagL, SpaL and MetS have been removed from plot (b), refer to text for full explantion (section 4.4.4)

The data lead to possible explanations for the anomalous behaviour of the magnesian limestone which has a much lower value for S than would be expected for its high μn_m. As noted earlier, n_e values were also substantially lower than those for n_t and n_m. The very large pores and cavities present on the surface of this rock allow it to drain very freely and rapidly and it is therefore possible that M_s (saturated mass) values used for the determination of n_e are underestimated. However, there are two reasons for rejecting this argument: (i) The freely draining nature of this rock was known prior to measurements being undertaken, and therefore, particularly strenuous efforts were made to minimise its potential effects. For example, the time that specimens spent in air during transfer from the water bath to the weighing apparatus was absolutely minimal. Also, a damp, non-absorbent chamois leather was always used for surface drying. (ii) Identical procedures were carried out for determination of n_t following vacuum saturation, values for which are comparable to those obtained by mercury impregnation. It is difficult to see how surface drainage alone could produce n_e values of around 50% those of n_t.

An alternate explanation might relate to the unusual pore size distribution for MagL, which despite a microporosity of 75%, also showed a particularly large proportion of coarse pores. For example, it ranks 4th for percentage of pores >5μm and 2nd for pores >10μm. This rock also shows, distinctively, a much greater variation in macropore sizes than the other rocks tested and a much narrower range of micropore sizes. This might be an indication that the relationship between microporosity and S does not just depend upon the percentage of pores above or below some arbitrary boundary, but on the more precise range and distribution of fine and coarse pores, and on their actual position in relation to each other. For instance, it might be that the suction effect of micropores would be to some extent negated if those fine pores were surrounded by coarse pores. These suggestions might have implications for interpretation of the much greater resistance to weathering shown by MagL than would be expected given its rock properties.

ROCK TYPE	ρ	W_{ab}	n_e	n_t	S_1	S_2	S_3	μn_m	n_m
LdCh (15)*[1]	1.74	19.07	33.24	37.14	0.93	1.00	0.97	65.58	38.58
MagL (10)	1.62	8.90	14.42	35.67	0.52	0.56	0.54	75.27	33.61
OolL (11)	2.16	7.88	17.02	19.27	0.94	0.98	0.96	93.18	7.62
HdCh (19)	2.01	10.98	22.04	26.03	0.90	1.00	0.95	99.59	24.23
SpaL (10)	2.66	0.18	0.47	0.54	1.24	1.00	1.12	18.18	0.60
WeaS (10)	2.23	4.84	10.78	15.23	0.85	0.73	0.79	19.87	14.17
CalS (14)	1.95	9.30	18.16	22.66	0.80	0.88	0.84	29.42	26.15
MicS (10)	2.09	7.43	15.55	19.55	0.76	0.86	0.81	35.38	12.53
LamZ (18)	2.52	2.71	6.84	7.46	1.01	1.07	1.04	97.58	5.39
MetS (10)	2.65	0.52	1.39	2.94	1.04	1.00	1.02	0.00	0.37

Table 4.7 Pre-test void-dependent rock properties

Where ρ = dry density g/cm^3; W_{ab} = water absorption capacity %; n_e = effective porosity %; n_t = total connected porosity %; S_1 = saturation coefficient based on small test specimens; S_2 = saturation coefficient based on standard test specimens; S_3 = mean of S_1 and S_2; μn_m = microporosity %; n_m = total connected porosity from mercury intrusion.

Note 1 The number of specimens upon which each mean is based is given in parenthesis in column 1.

4.4.6 Micro-structure

Scanning Electron Microscope analysis has been used to obtain micro-structural information about each rock type and to verify the indirect pore size data obtained from mercury intrusion porosimetry (Plates 4.1 to 4.5 on pages 98 to 102). Rock types are described in approximately ascending order of pore size as determined by ranking data obtained from pore size distribution (Table 4.8). This ranking considers mean, median and modal pore throat diameter.

ROCK TYPE	*Mean* p.t.d*	Rank	*Median* p.t.d	Rank	*Modal*[*2] p.t.d	Rank	Overall Ranking
LdCh	0.6775	8	0.9252	5	1.0143	5	**7**
MagL	0.4402	7	0.4177	4	0.3533	2	**4**
OolL	0.1989	4	0.3554	2	0.5702	4	**2**
HdCh	0.2961	5	0.4014	3	0.4058	3	**3**
SpaL	1.1212	9	13.1180	9	3.5145	8	**9**
WeaS	0.3820	6	10.6064	8	1.7829	6	**8**
CalS	0.1555	3	3.0591	7	2.9947	7	**6**
MicS	0.1232	2	2.7025	6	7.4786	9	**5**
LamZ	0.0139	1	0.0176	1	0.0141	1	**1**
MetS	43.6353	10	48.5223	10	43.2507	10	**10**

Table 4.8 Ranking of rock types according to pore throat diameter data

Note 1 p.t.d = pore throat diameter

Note 2 Modal p.t.d is determined graphically from pore size distribution plots given in Figures 5.17 to 5.26, mean and median values are calculated.

The laminated siltstone (LamZ) has a very fine grained, dense, generally tightly interlocked structure with a mixture of platey minerals and rounded detrital quartz grains (Plate 4.1a and b). Pores are irregular in shape and generally less than 0.5μm in diameter, though some pores 2-3μm occur. Pores exceeding 10μm are occasionally in evidence, generally associated with intergranular spaces between coarse grains of >15μm (Plate 4.1b). Also, occasional, well developed rounded pores of 20μm occur (Plate 4.1c). The high rock density is achieved due to the wide range of grain sizes present, in which fines infill gaps between coarser grains. These observations coincide well with the bi-modal pore size distribution obtained from mercury porosimetry.

There are also several distinctive sizes of pore present in the coarse-grained oolitic limestone (Plate 4.1d) (OolL). Macropores of >30μm (>1000μm^2 in area) are formed in the irregular spaces between concentric oolith layers (Plate 4.1e). However, the layers of the ooliths themselves comprise a much more regular and uniform, though tightly packed pore structure, with diameters up to 10μm (Plate 4.1f). The very fine pores of the laminated siltstone are absent (Plate 4.1f). Most of these smaller pores are intergranular with very tight throats, almost appearing to be isolated, while others are more akin to cracks and have developed in association with fossils (Plate 4.1g). Spatial variations in porosity are apparent, varying from extremely tightly packed to very open (Plate 4.1h). A tightly packed area is seen in the bottom half of the micrograph in Plate 4.1h, in which porosity is very low and pores are small. In the upper half, grains are much more loosely packed, relatively uniform in size, with good connectivity between pores. The difference might reflect variation in precipitation of cementing materials in void spaces and could explain the large apparent difference between pre-test values for n_e and n_m.

The high density chalk (HdCh) has a well packed, blocky, micritic structure with crystals typically <3μm, and occasional authigenic calcite crystals of 20-30μm. There are many coccoliths and fossil fragments present (Plate 4.2a). There is also a moderate degree of interlocking between grains, though in more porous areas associated with foraminifera grains appear isolated (Plate 4.2b). Pore sizes generally occur in a very narrow range of less than <1μm, with narrow connections (Plate 4.2c). Rarely, narrow macropores occur as non-persistent grain boundary cracks associated with platey minerals and shell fragments (Plate 4.2d). The very narrow range of pore sizes observed here is reflected well in the pore size distribution curve given in Figure 4.15.

The magnesian limestone (MagL) is also blocky and micritic but with less interlocking than the high density chalk, and a consequently higher porosity. Two distinct groups of macropores occur. The first group consists of very large macropores of 40 to 100μm and occasionally up to 300μm. These are well rounded (Plate 4.2e), isolated pores with extremely narrow pore throats if any connections exist at all. These pores are sometimes connected by narrow inter and intra-granular cracks of 2-3μm, and occasionally contain authigenic crystals (Plate 4.2f). The second group of pores lie in the range 2 to 10μm. These have better connections than the larger pores, though are somewhat irregular. Mostly they are moderately equidimensional voids between grains, while sometimes they form from intragranular cracks (Plate 4.2g). MagL also contains a large proportion of micropores and many pores around 1μm and less can be seen in Plate 4.2h. SEM analysis clearly supports the findings of mercury intrusion porosimetry in which distinct populations of both micro and macro pores were indicated.

The micaceous sandstone (MicS) consists of large, rounded, detrital grains of quartz typically 50μm in size, with a fine intergranular matrix of particles up to 5μm with platey minerals interspersed (Plate 4.3a). There are also some large authigenic crystals of quartz (Plate 4.3b) which often contain cracks. The result is that a wide range of pore sizes is present, as indicated in the pore size distribution (Figure 4.15). Some of these are large pores of around 70μm (Plate 4.3c) associated with the larger grains, some are micropores in the rock matrix and others are a mixture of intragranular macropores and linear pores associated with the micas (Plate 4.3d). Rarely, there are also isolated pores of 25μm (Plate 4.3e) and some grain boundary cracks associated with detrital grains.

The calcareous sandstone (CalS) contains well rounded detrital quartz grains of 100μm (Plate 4.3f) in a fine calcite and quartz rich matrix of 1 to 5μm (Plate 4.3g). There is minimal interlocking of grains and the texture is slightly loosely packed. The matrix contains a wide range of pore sizes from less than <1μm and up to 3μm, but with macropores of 50 to 60μm also being common. These pores show evidence of coalescence, and bridges between pores have sometimes been breached, perhaps by dissolution of the calcite matrix. Some microcracks exist around larger grains and there are also several larger inter and intra-granular, angular cracks (Plate 4.3h). The actual distribution is again well reflected in the mercury porosimetry distribution.

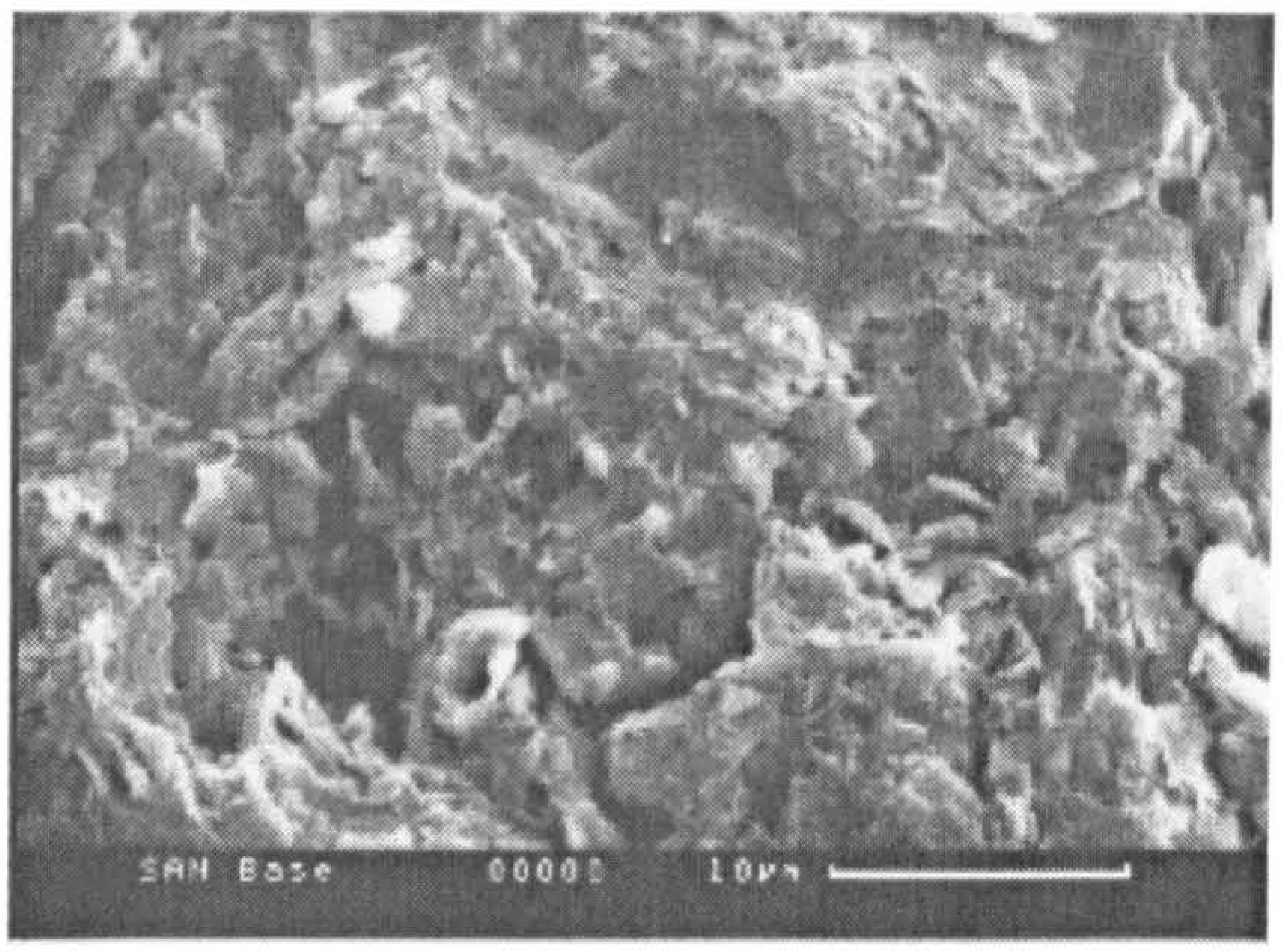
The low density chalk (LdCh) has a similar blocky, micritic structure to the high density chalk, and contains fossils, shell fragments and coccoliths. Interlocking is absent however, and grains, typically around 3µm, are isolated, almost appearing to 'float' (Plate 4.4a), and the structure is very open. The uniform grain size leads to pore sizes occurring in a narrow range of around 1µm, a little larger than found in HdCh. As with HdCh, areas of greater porosity and larger pores of 8 to 30µm and rarely 150-220µm occur in association with fossils (Plate 4.4b and c). Occasionally there are lower porosity areas where grains are tightly packed (Plate 4.4d). Rarely there are 30µm diameter spar-lined foraminifera tests which appear unconnected to other spaces. Occasional microcracks can also be observed.

There is a wide range of pore sizes in the weathered sandstone (WeaS) and this is probably partly due to the wide grain size range represented. There are two distinct populations of macropores. One group consists of angular pores of 30µm diameter within authigenic quartz overgrowths at the three-way junction between detrital quartz grains (Plate 4.4e). These also occur with angular, intragranular cracks of aperture 8-20µm which are due to shattering of the authigenic crystals. The second group consists of rounded macropores between detrital grains typically of diameter exceeding 100µm (Plate 4.4f). Irregular cracks open to 30µm also occur in association with these macropores (Plate 4.4g). It seems likely that the larger macropores represent junctions between detrital grains where authigenic quartz has not grown. There is also an inter-granular matrix porosity with pores open to 5-10µm and microcracks open to 2-3µm. These inter-matrix voids are larger than those for most other rocks, though some micropores also occur (Plate 4.4h). A further distinctive group of linear void spaces is associated with platey minerals (Plate 4.5a).

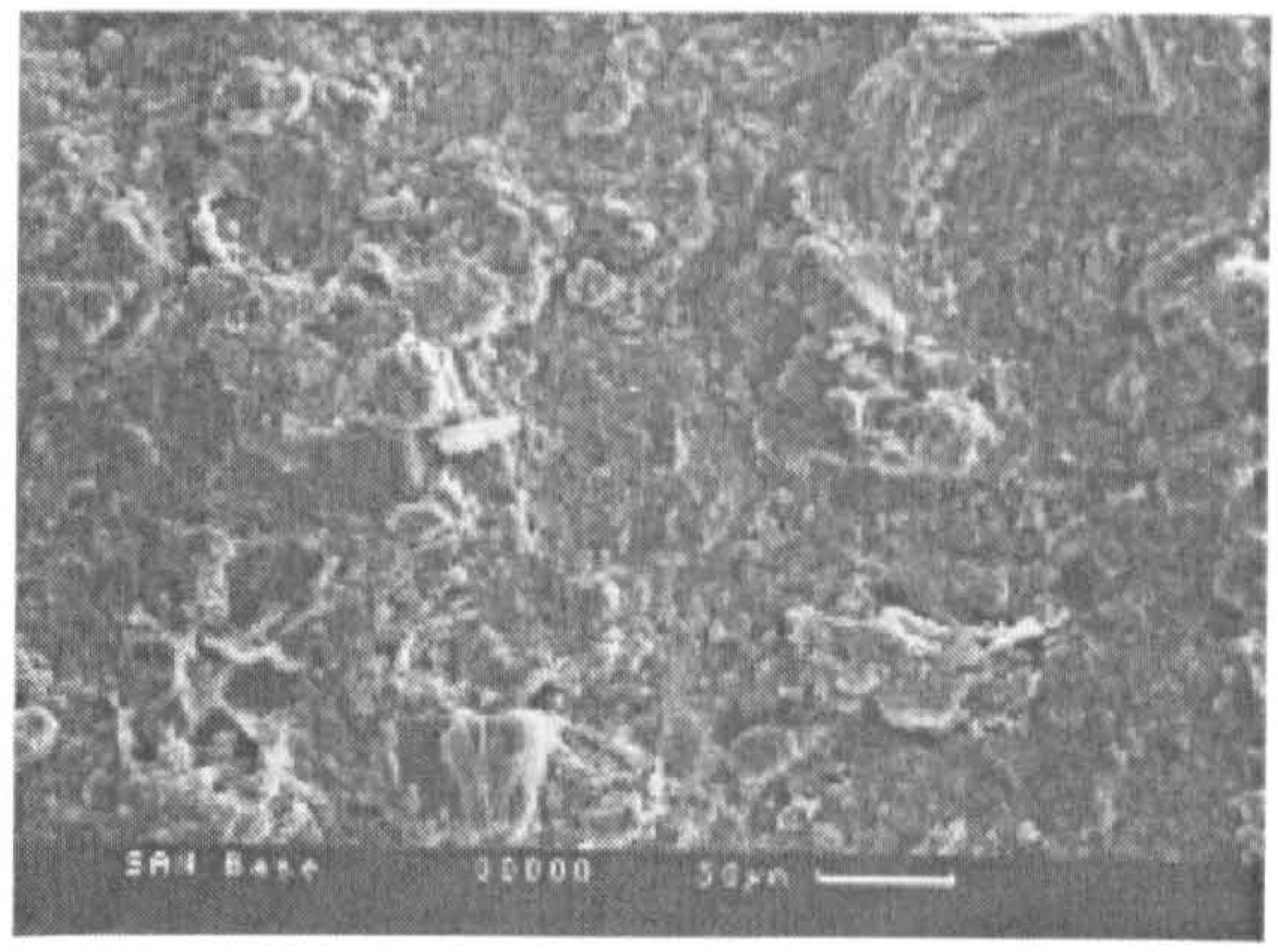
Mercury porosimetry data for the sparry limestone and the metasediment are unreliable due to the very low porosity of these rocks and therefore, SEM analysis offers a direct means of obtaining information on the actual pore structures present.

The sparry limestone (SpaL) is a tightly packed, interlocking material containing fine pores up to 2µm (Plate 4.5b) except in the vicinity of fossils where pores up to 25µm occur and the pore structure is much more loose (Plate 4.5c and d). From examination of micrograph (b) (Plate 4.5), it is suggested that total pore space could be as much as ~4%, but the proportion of this which is connected is very small and pore throats also appear to be extremely small. Connections between pores in the vicinity of fossils is probably much higher and pores up to 8µm occur, although the total amount of pore space is still low. Some fossils are infilled with both detrital and authigenic calcite. Microcracking occurs in association with larger calcite crystals and there are extremely rare, isolated surface pores of 60µm (Plate 4.5e).

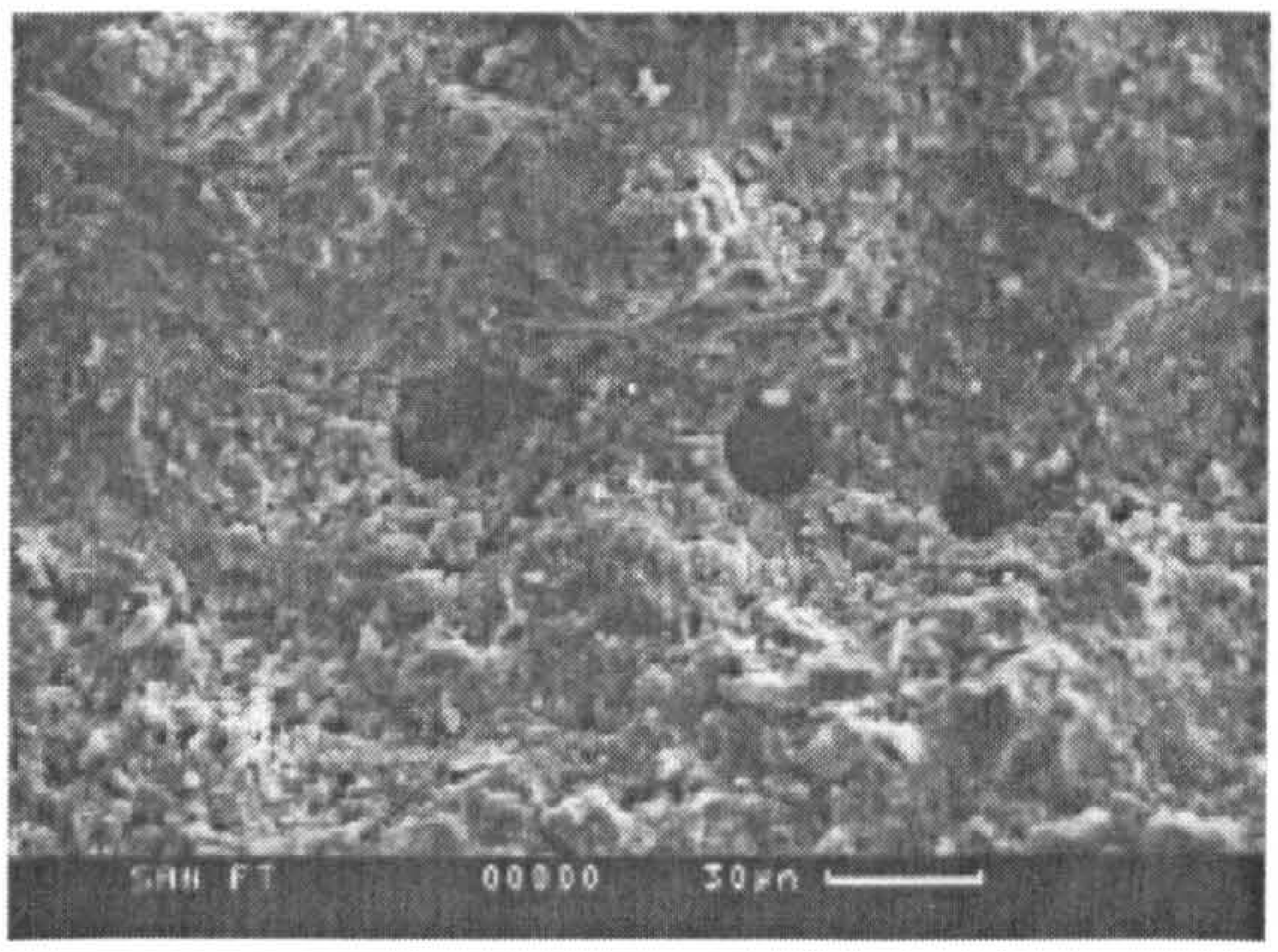
The metasediment (MetS) is extremely dense (Plate 4.5f) with an interlocking texture. Pores up to 10µm occur, but most are <5µm although a high percentage are <1µm and many of these appear to be unconnected (Plate 4.5g). The larger pores are associated with rare grain boundary cracks, aligned pores or platey minerals (Plate 4.5h). The small size of pores and their relative scarcity makes it difficult to assess the degree of connectivity between pores but it does not appear to be high.



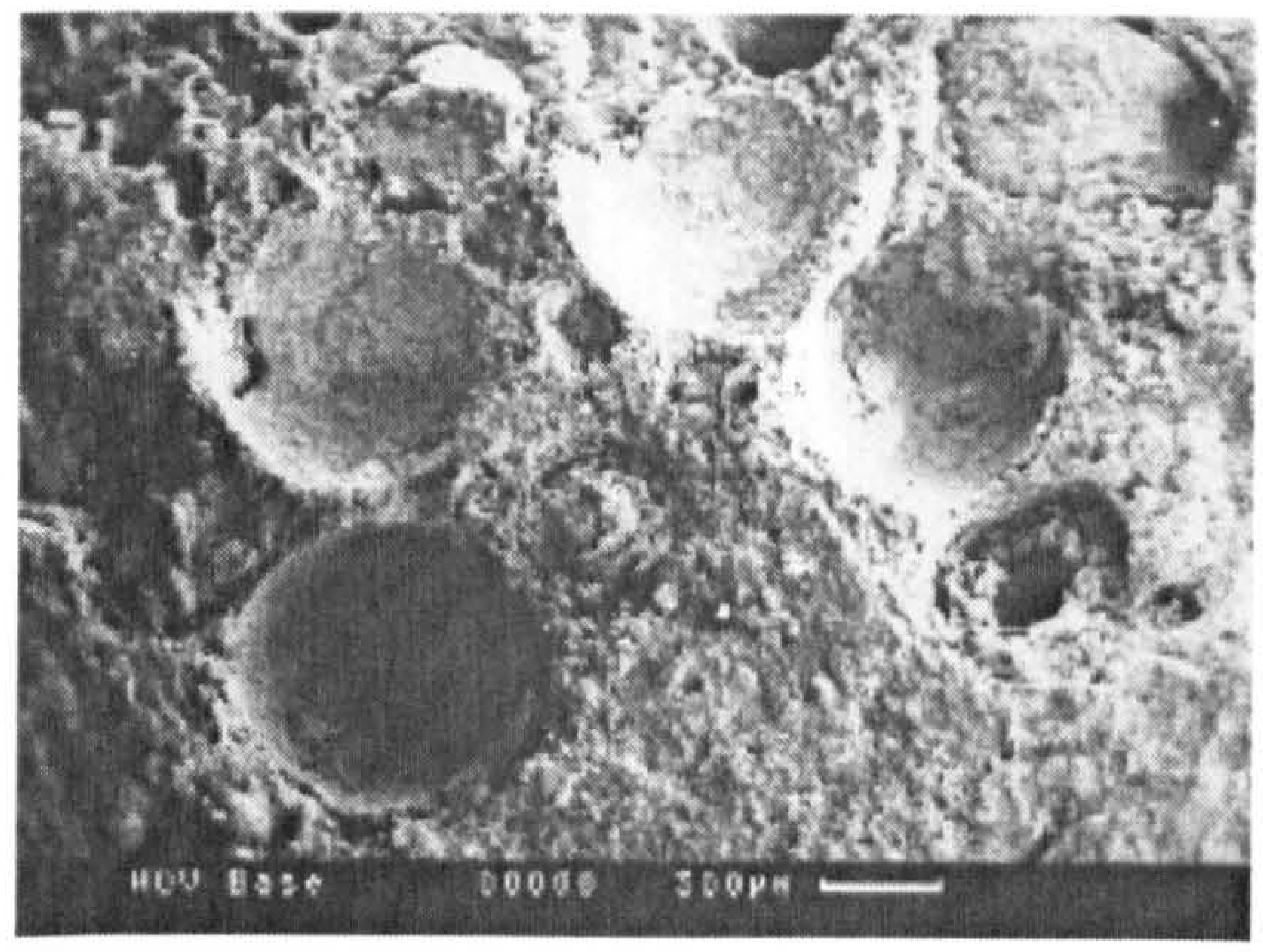
(a) Laminated siltstone (10μm)



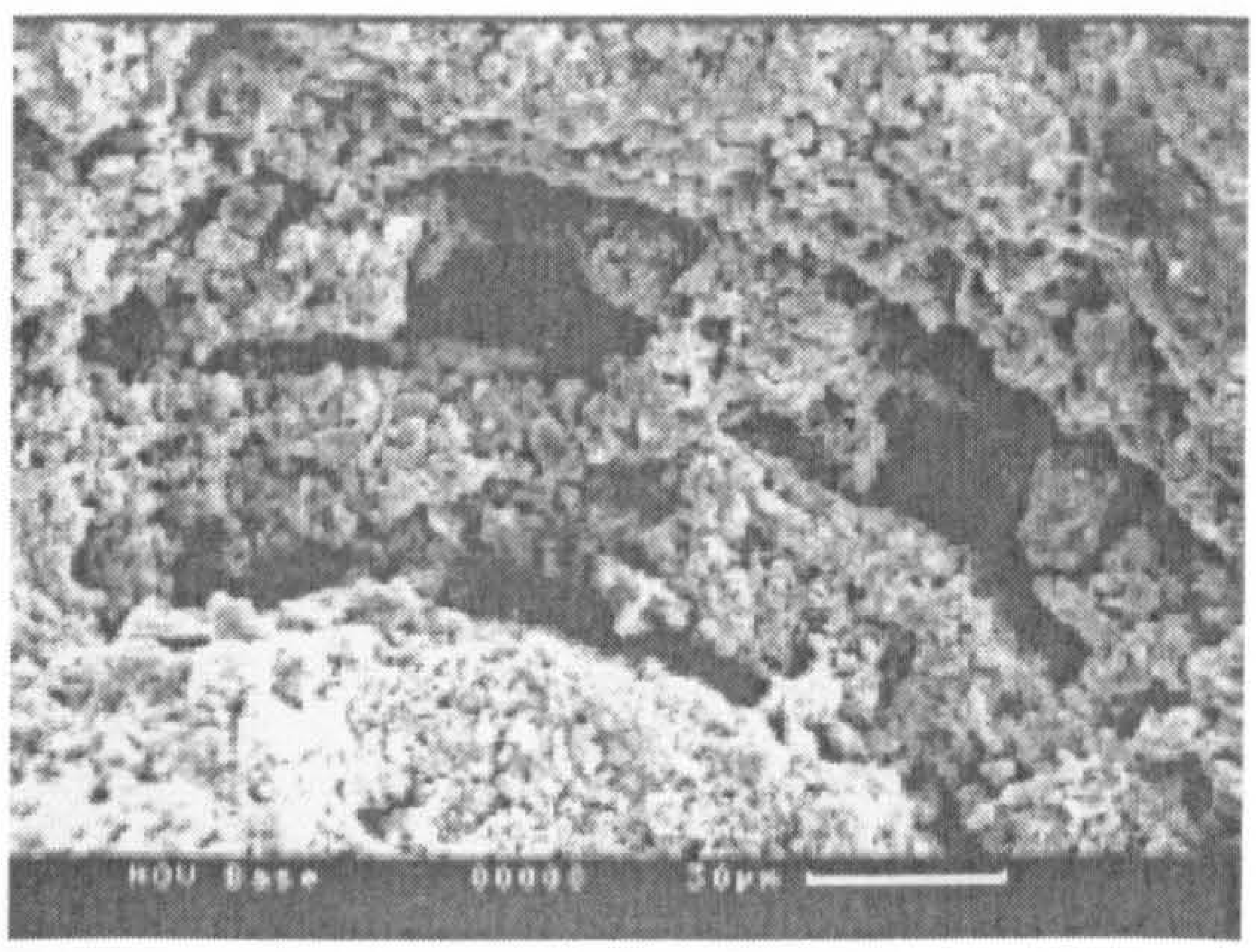
(b) Laminated siltstone (30μm)



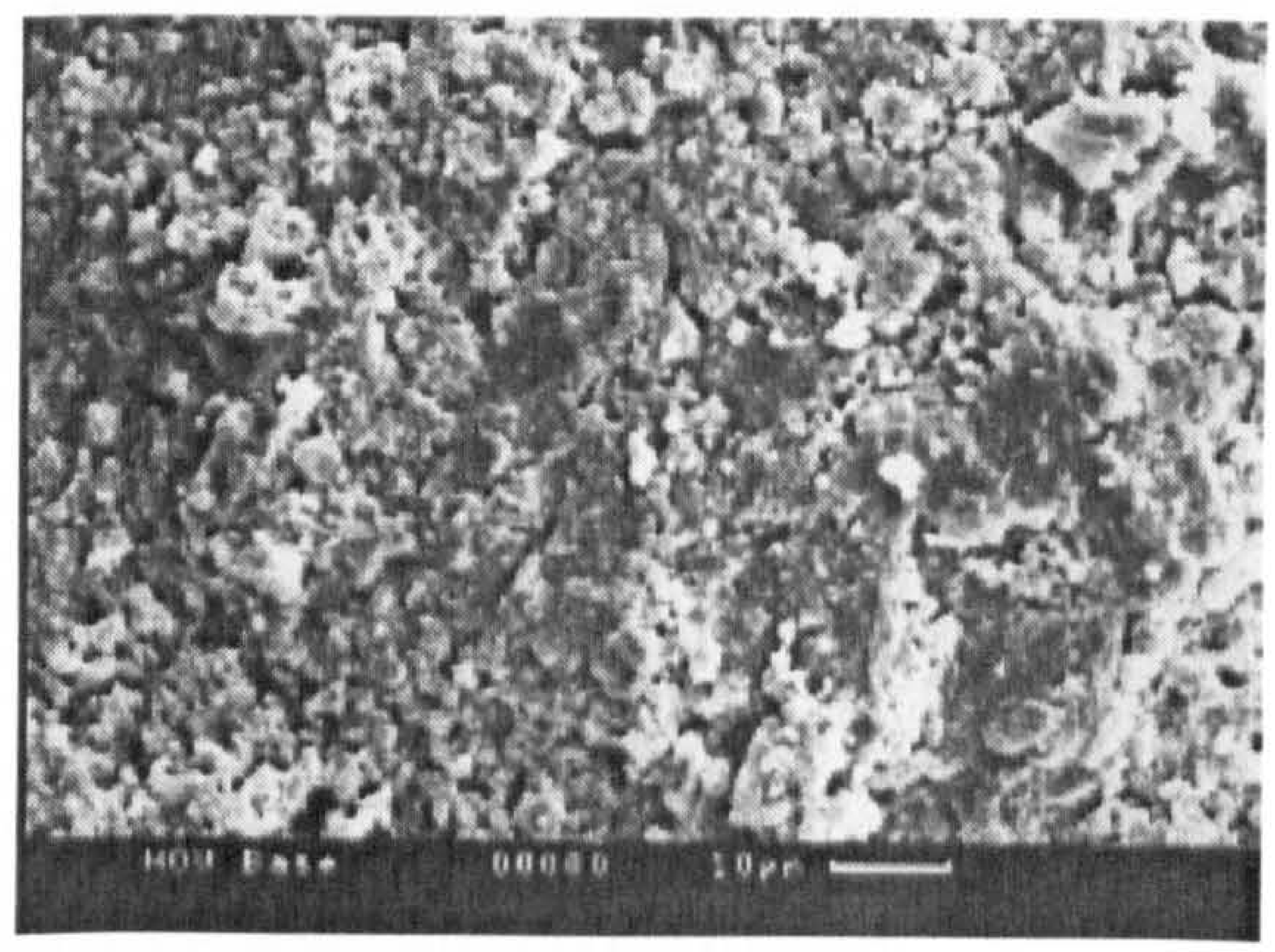
(c) Laminated siltstone (30μm)



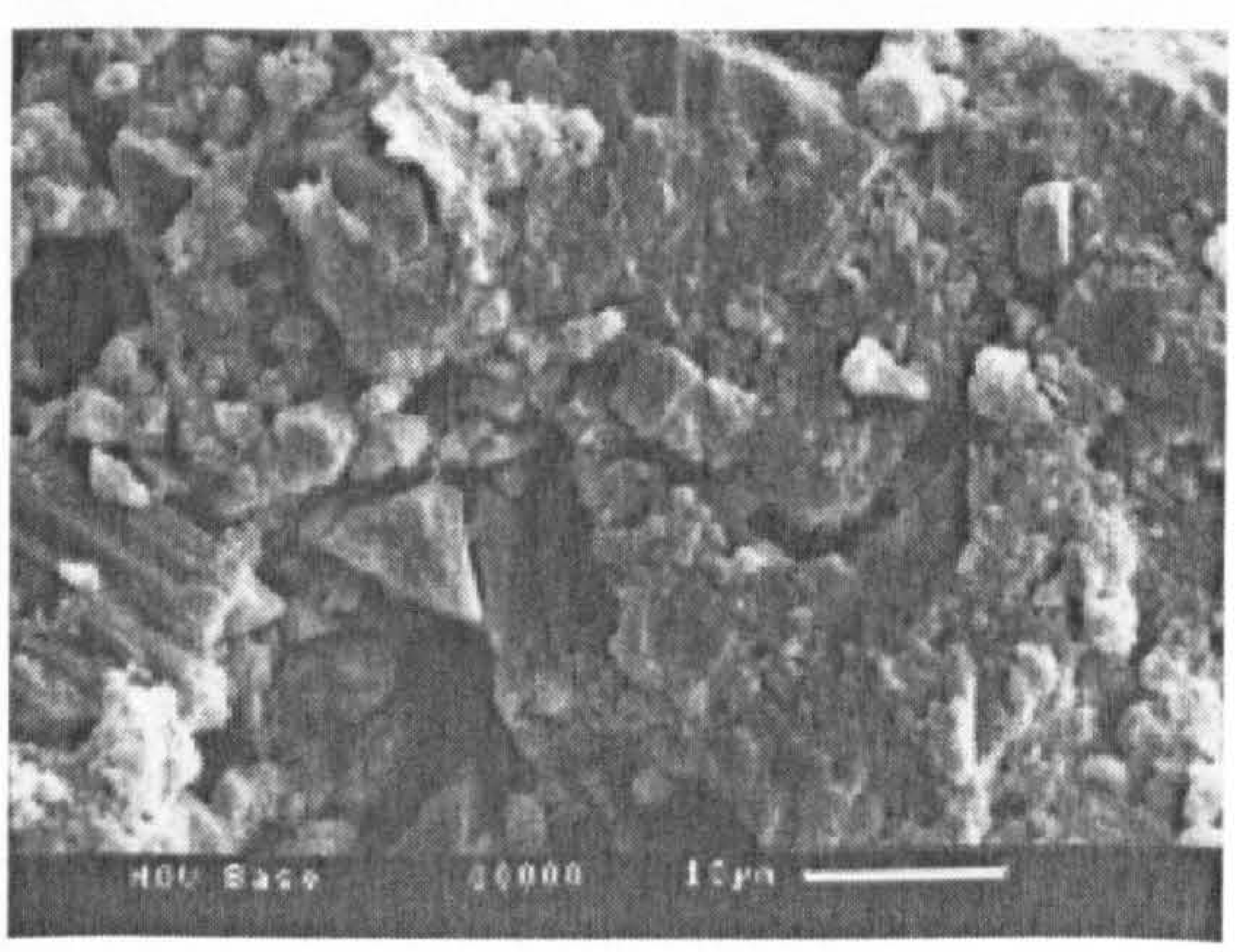
(d) Oolitic limestone (300μm)



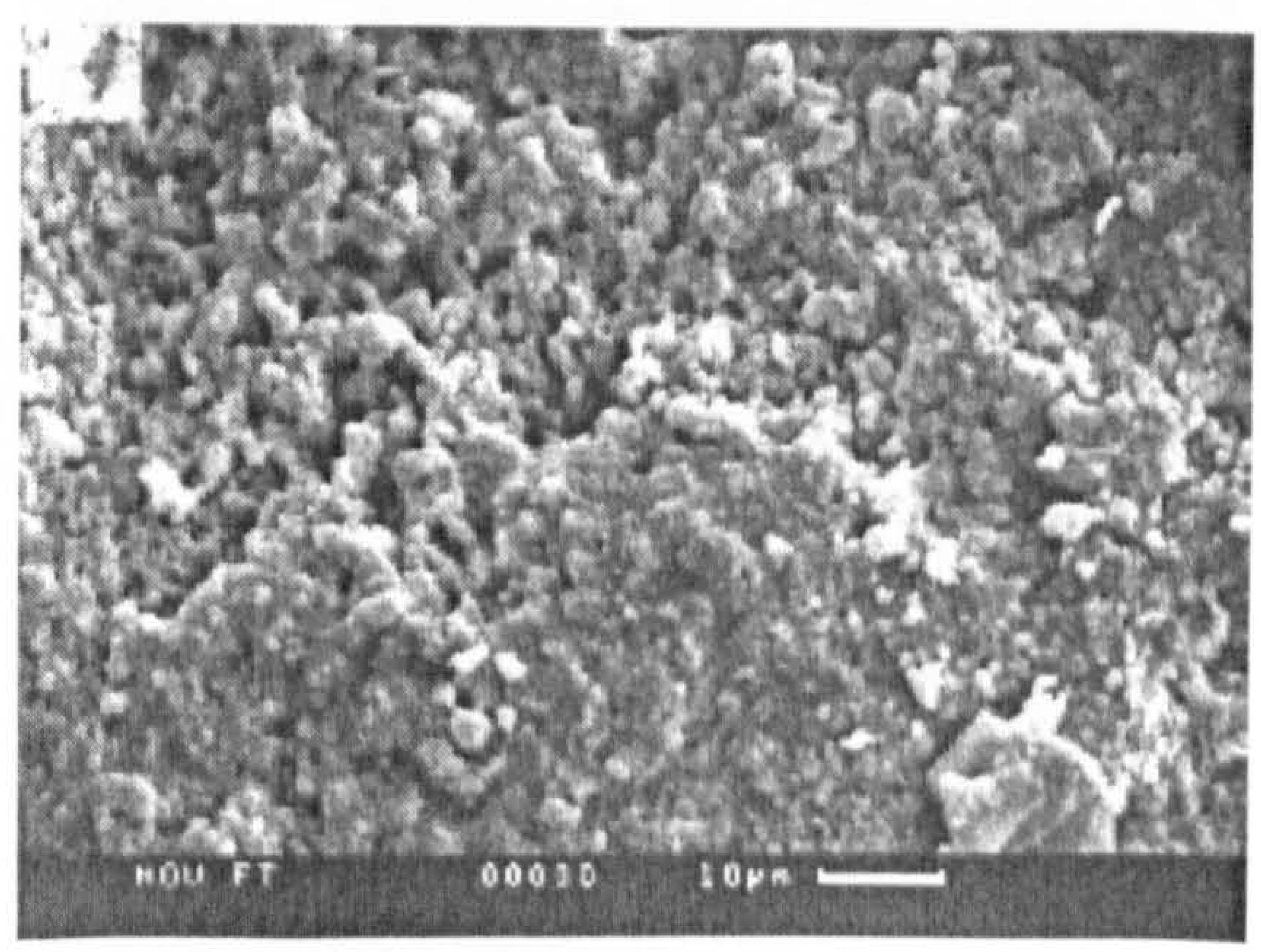
(e) Oolitic limestone (30μm)



(f) Oolitic limestone (30μm)



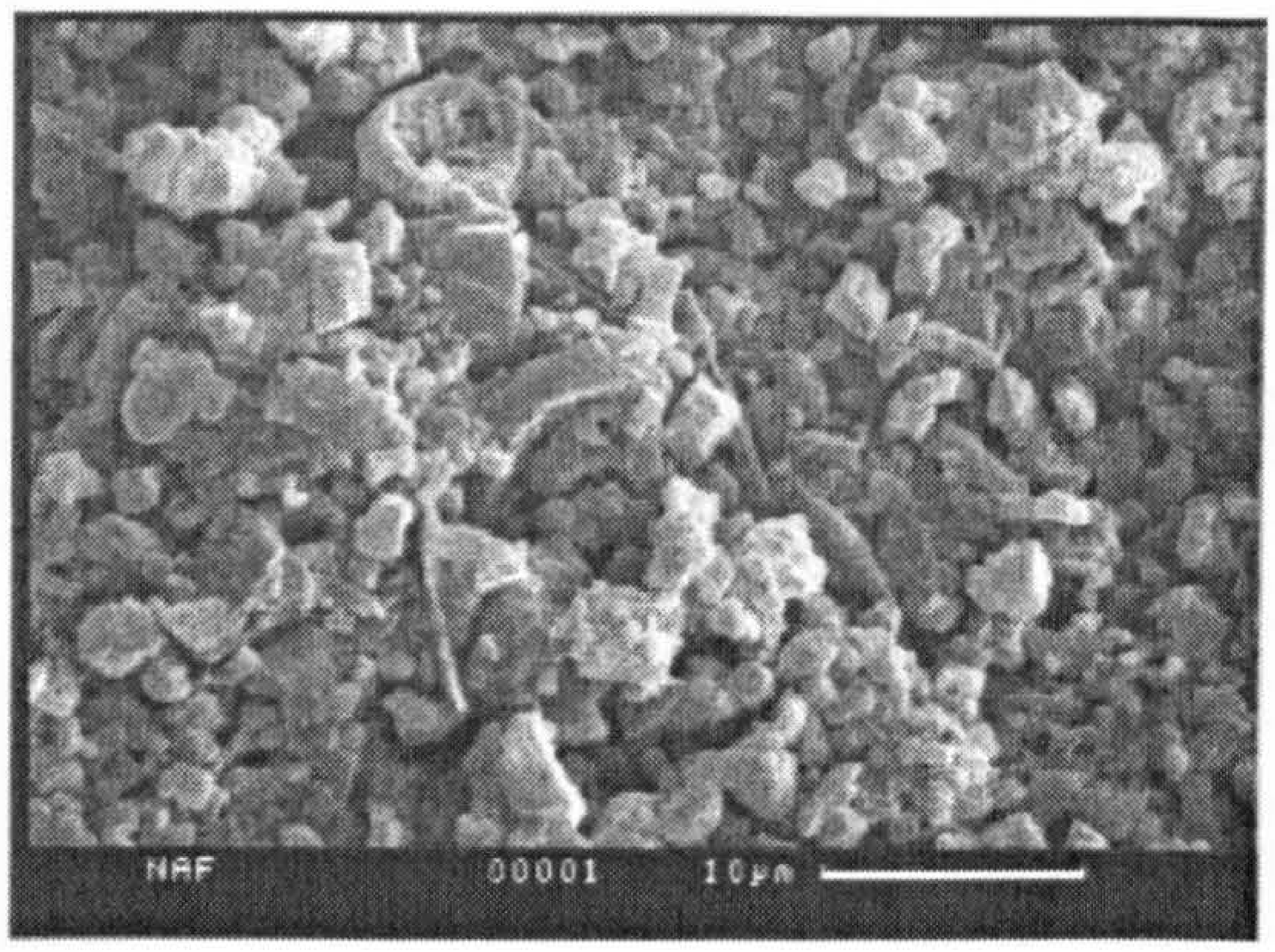
(g) Oolitic limestone (10μm)



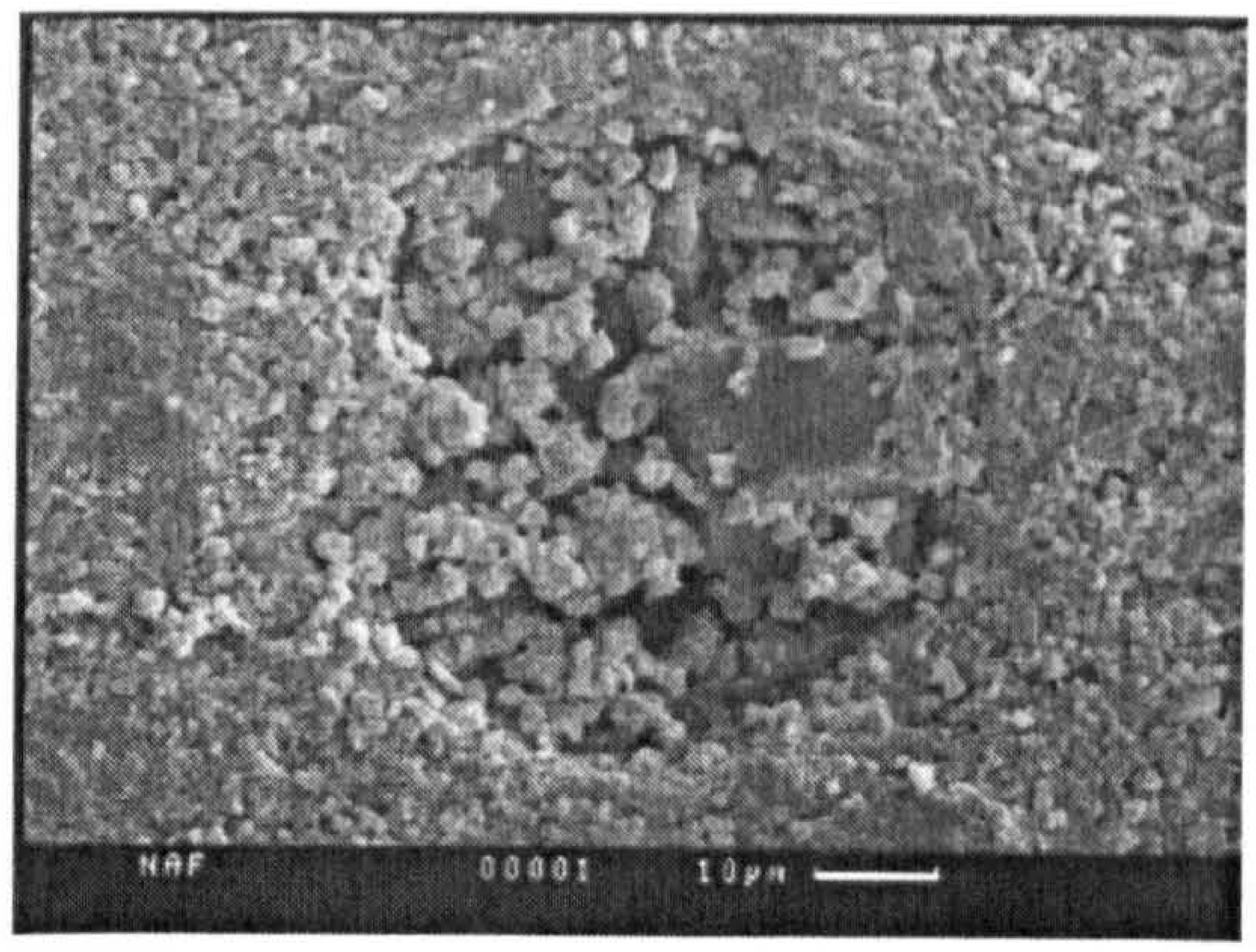
(h) Oolitic limestone (10μm)

Plate 4.1 Pre-test scanning electron micrographs

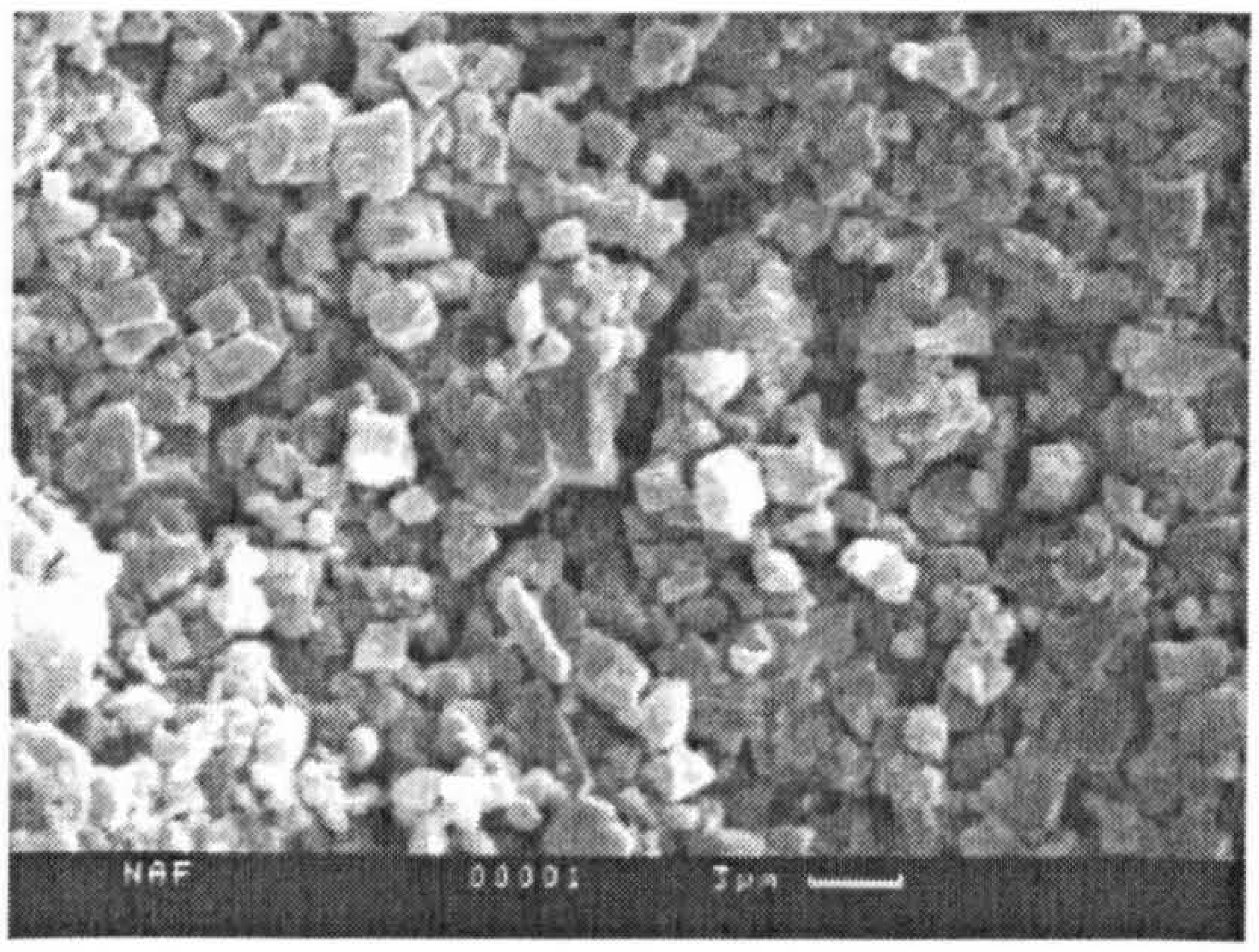
Refer to text for explanation (numbers in parenthesis give length of scale bar)



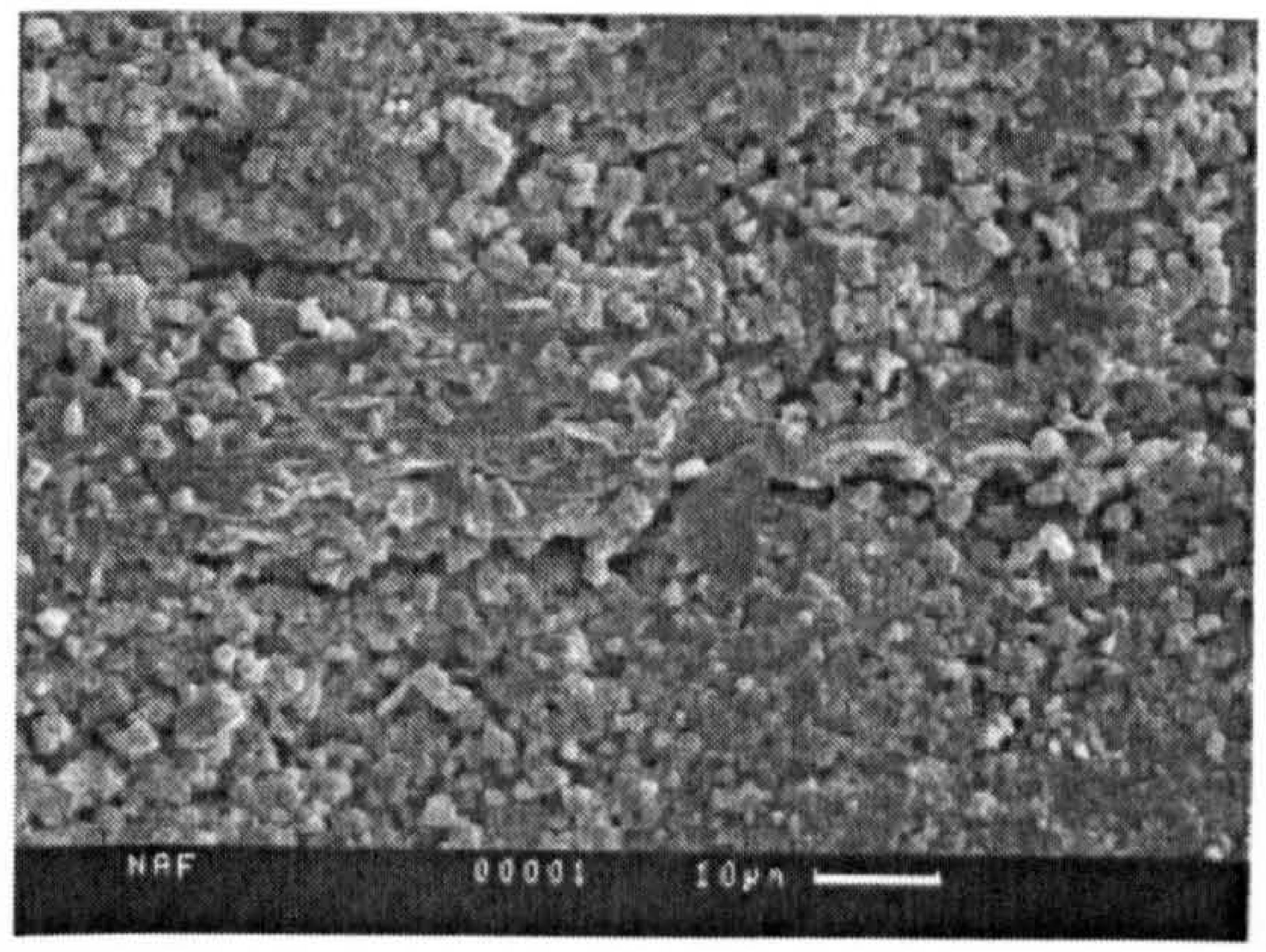
(a) High density chalk (10μm)



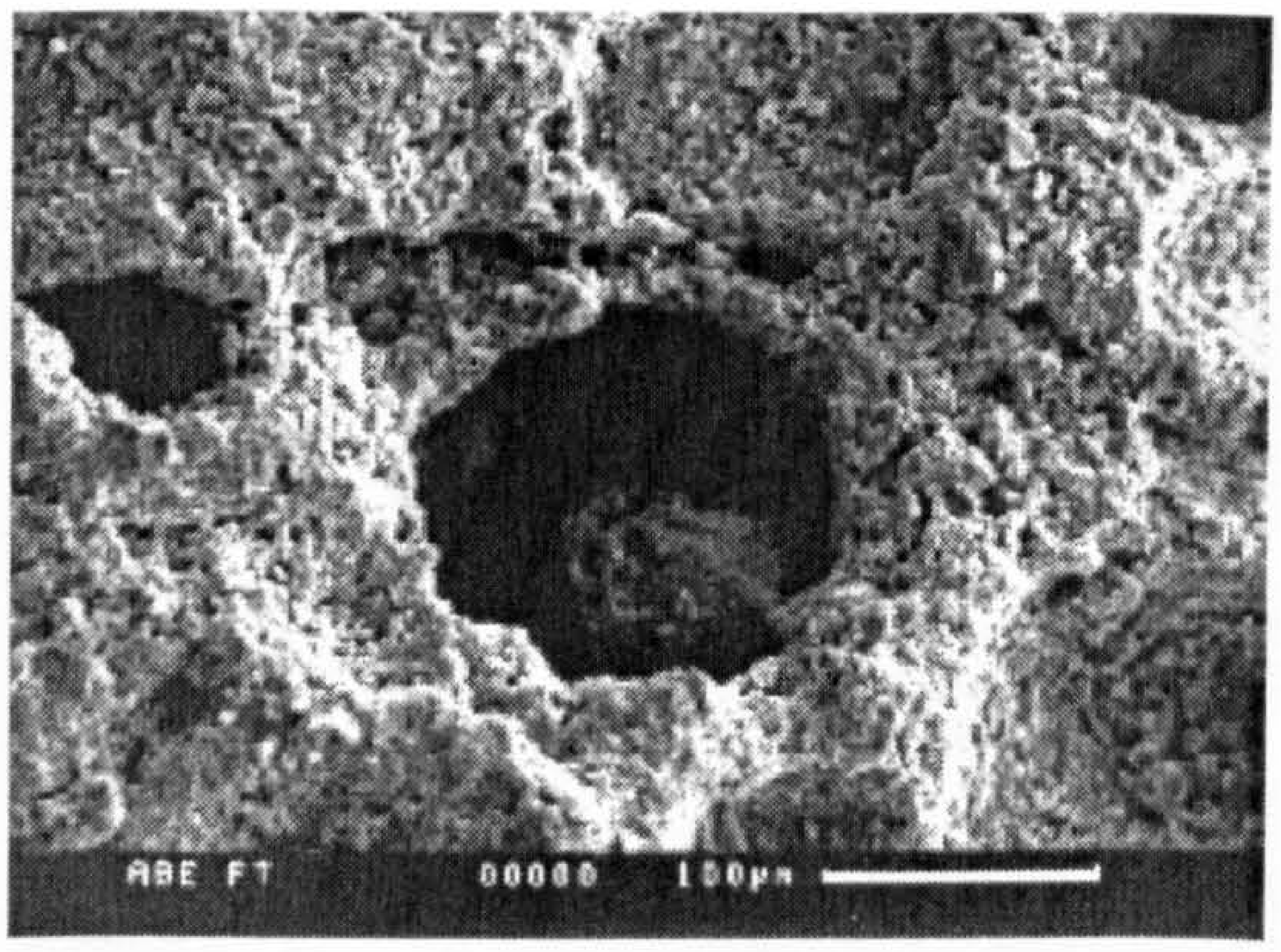
(b) High density chalk (10μm)



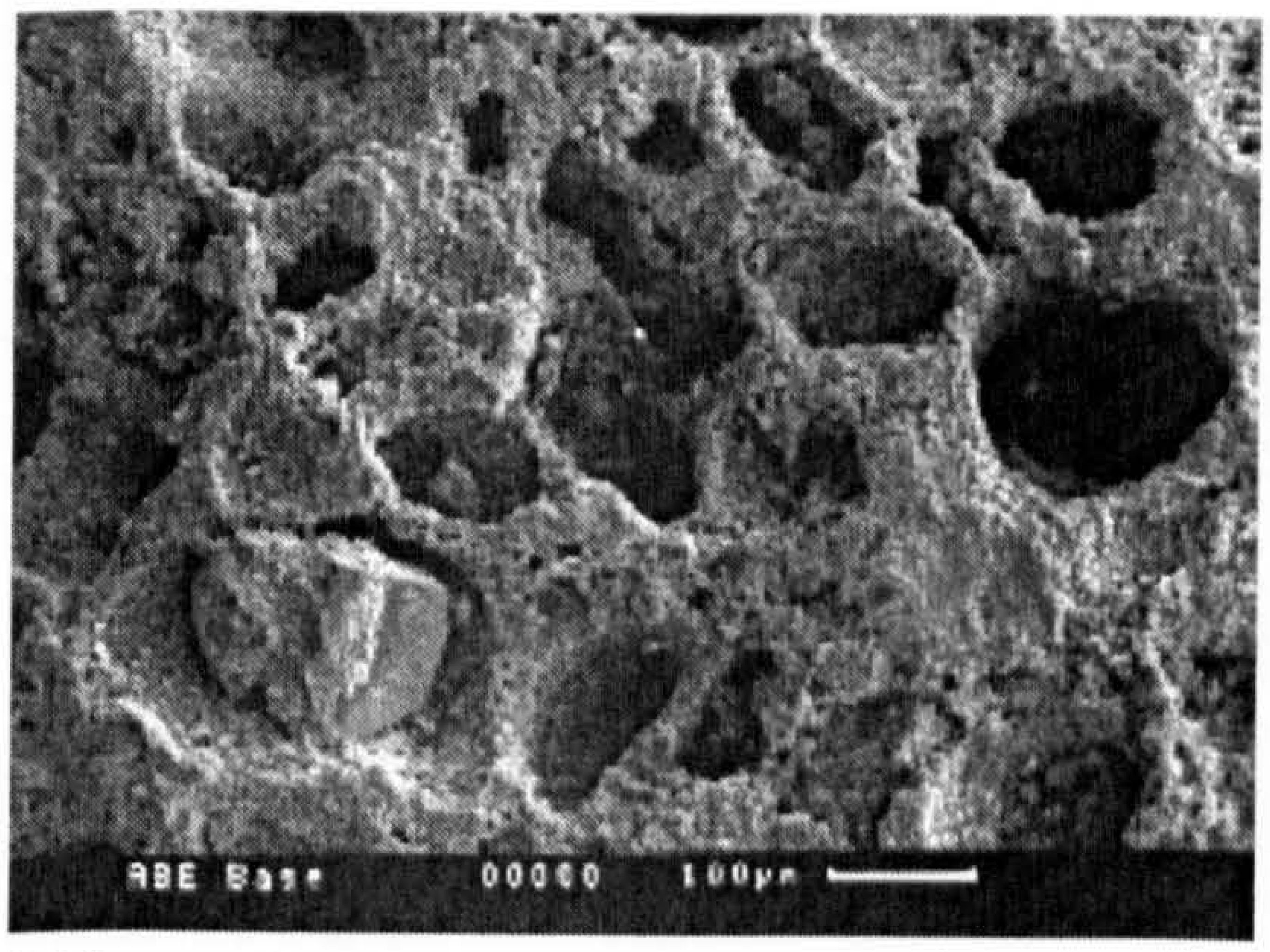
(c) High density chalk (3μm)



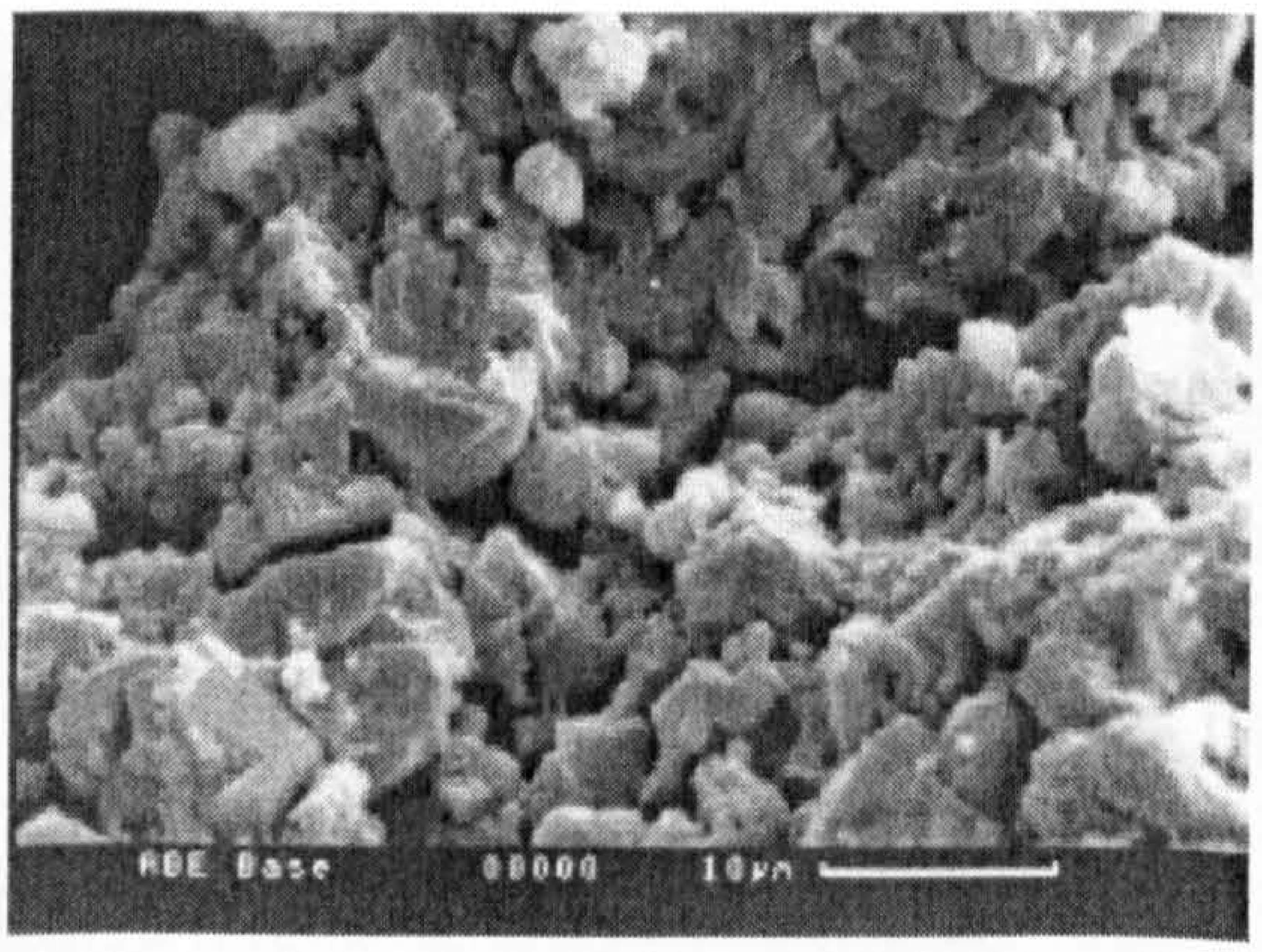
(d) High density chalk (10μm)



(e) Magnesian limestone (100μm)



(f) Magnesian limestone (100μm)



(g) Magnesian limestone (10μm)

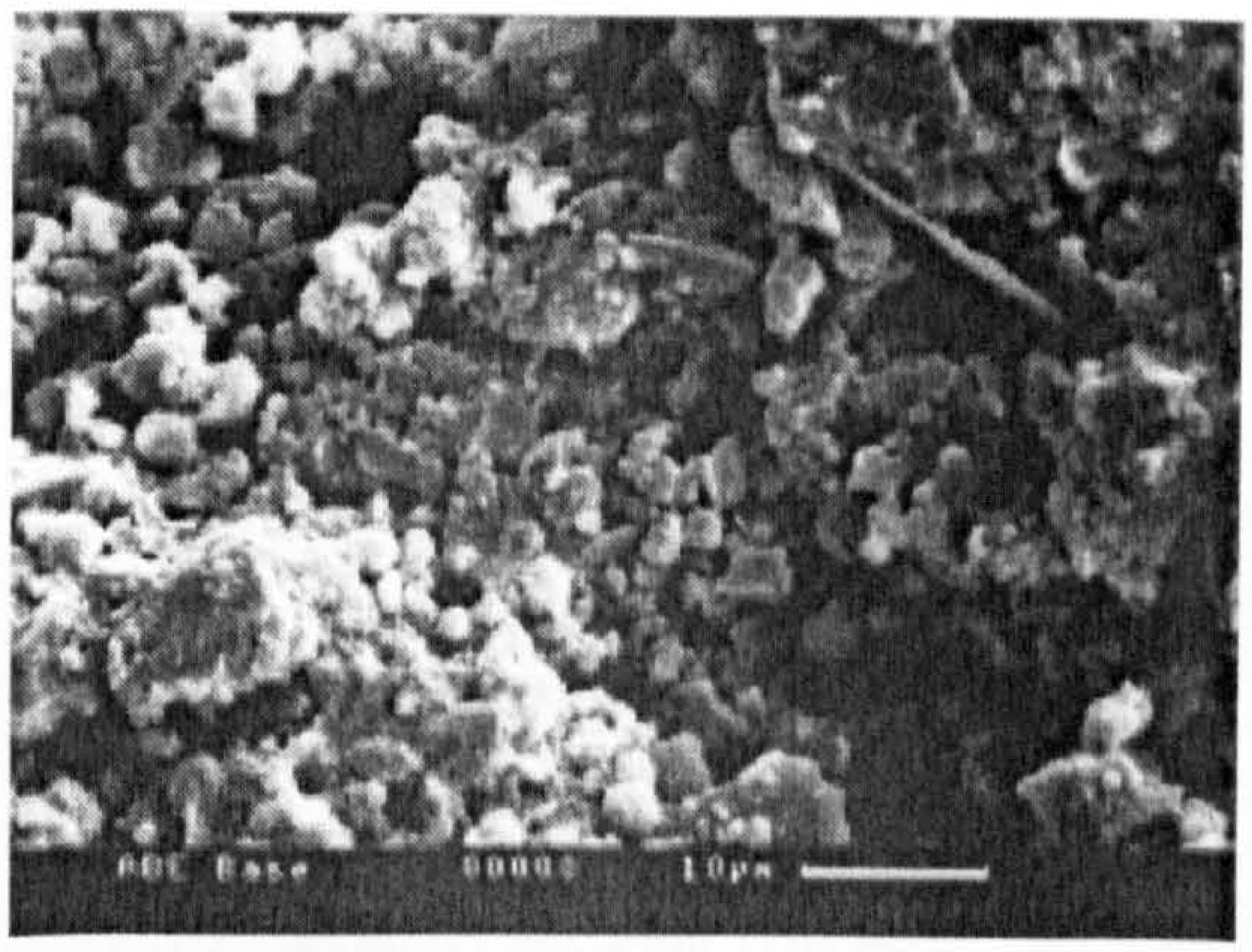
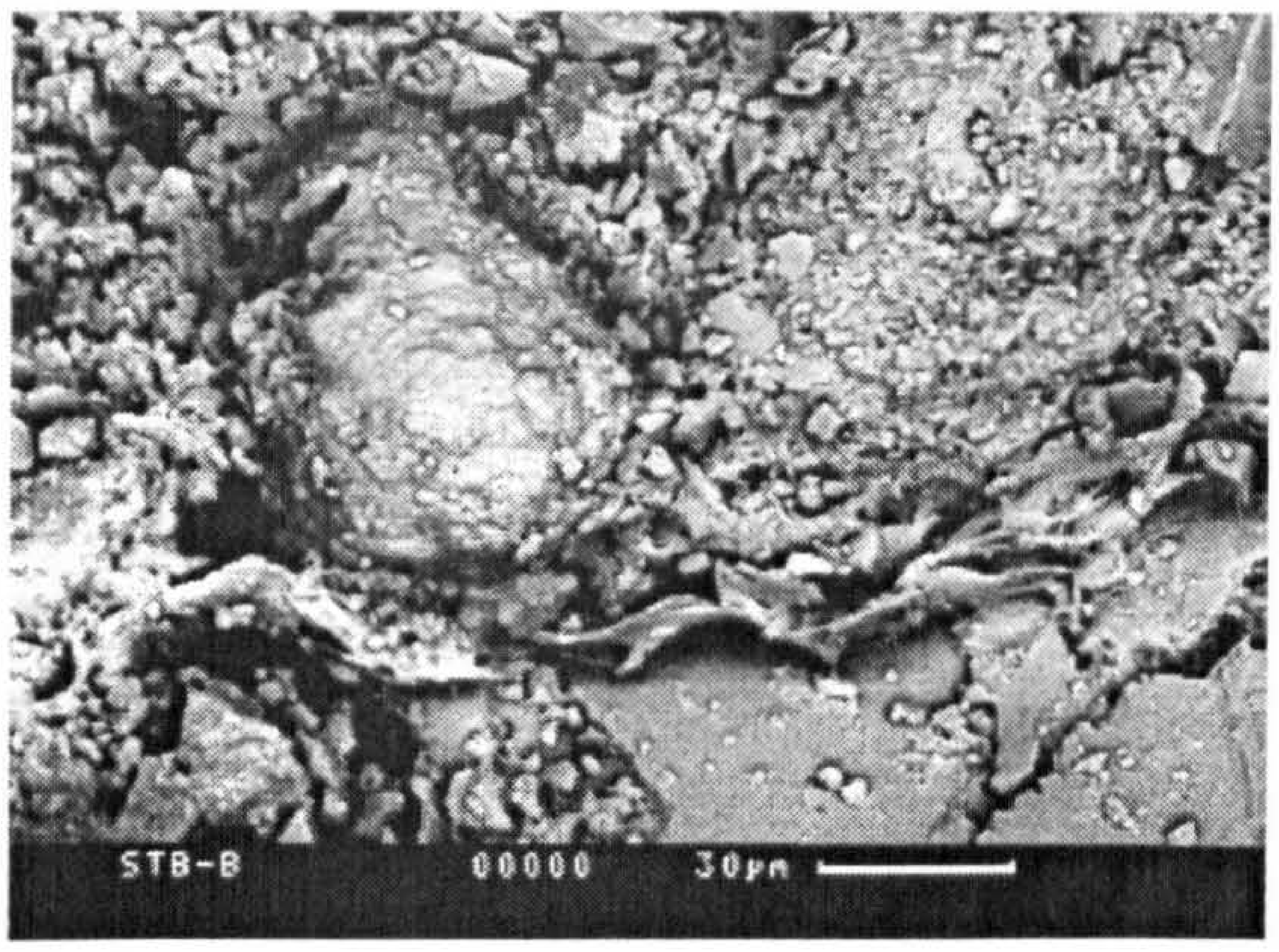
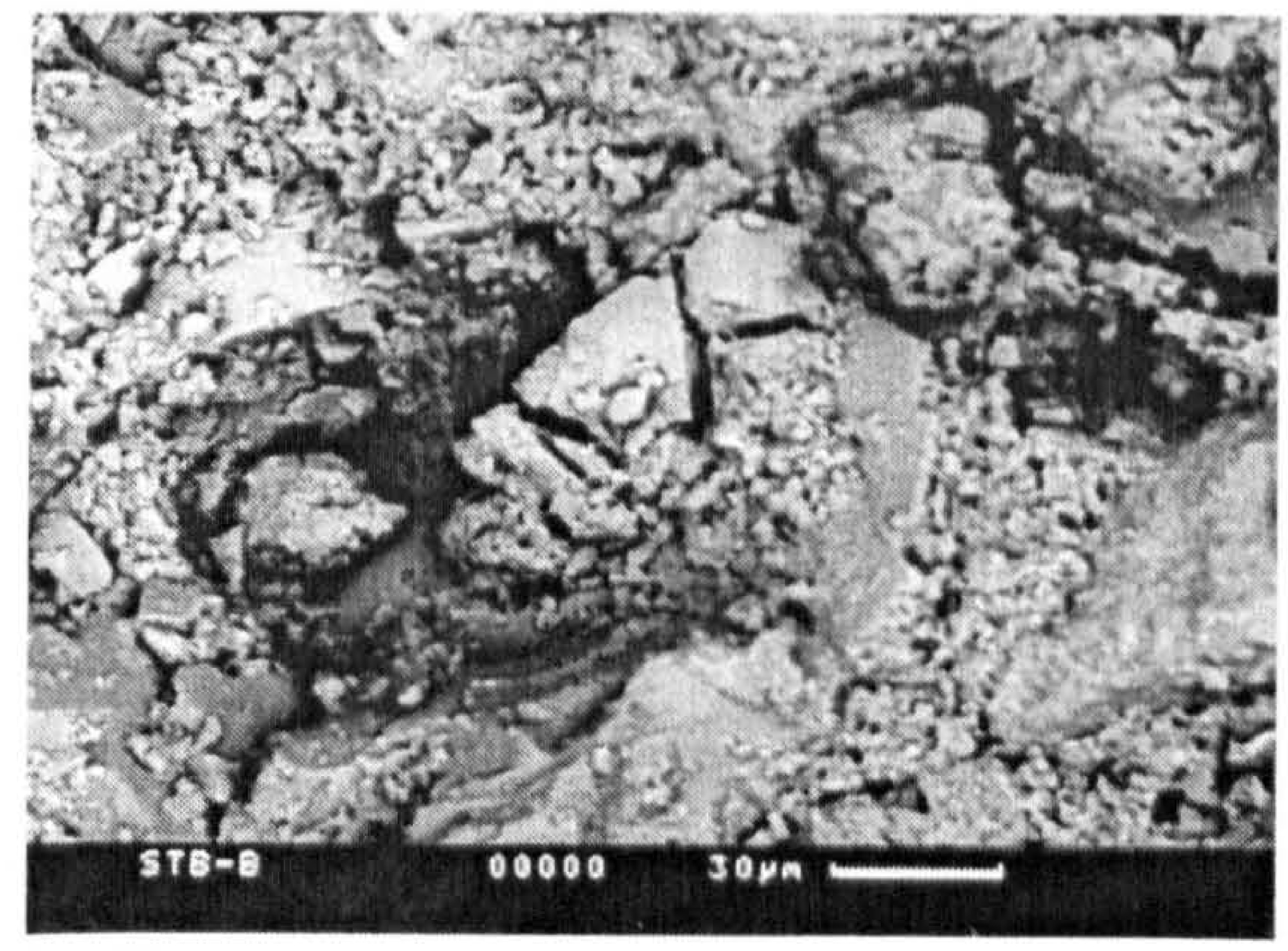

(h) Magnesian limestone (10μm)

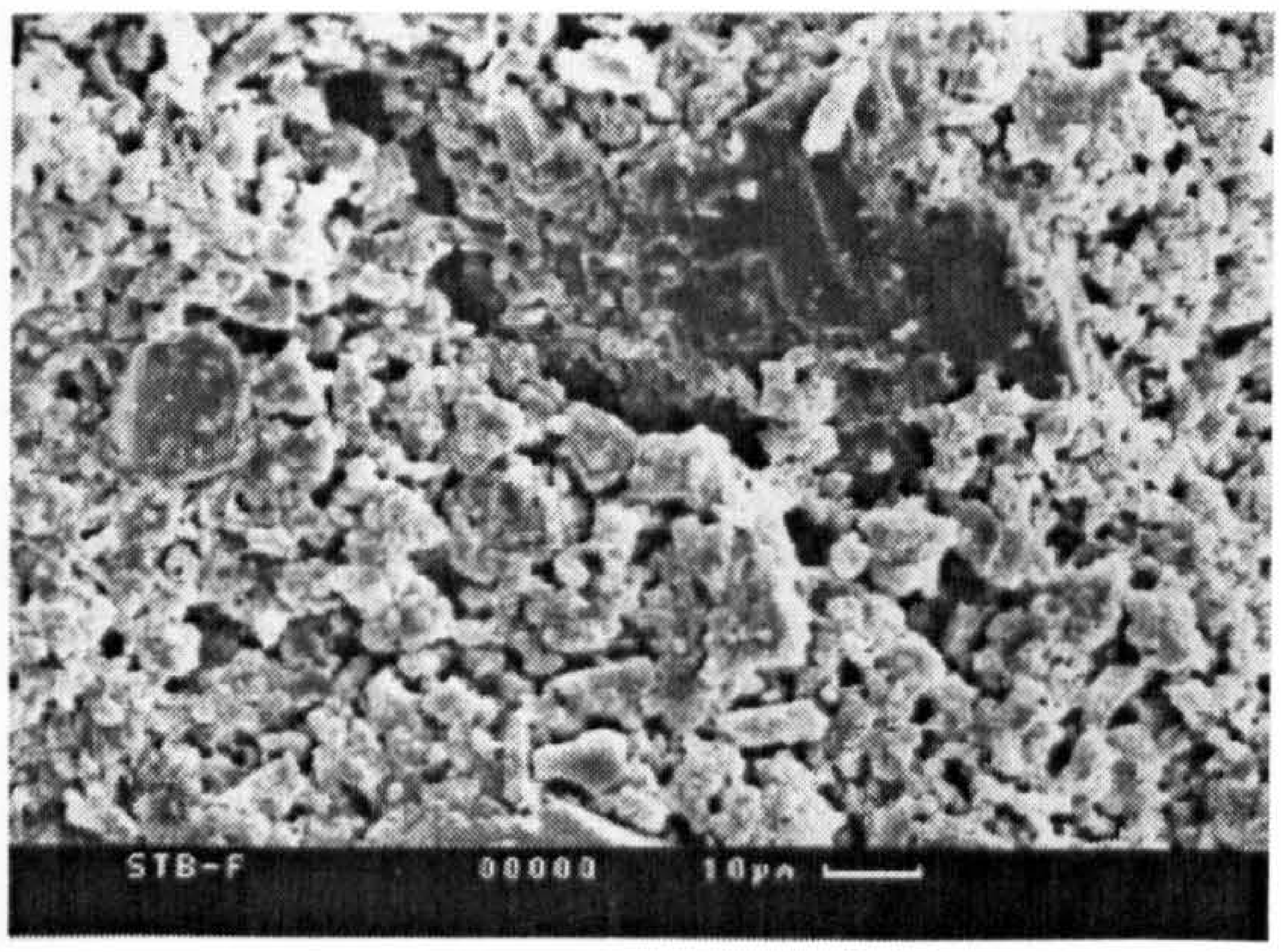
Plate 4.2 Pre-test scanning electron micrographs
Refer to text for explanation (numbers in parenthesis give length of scale bar)



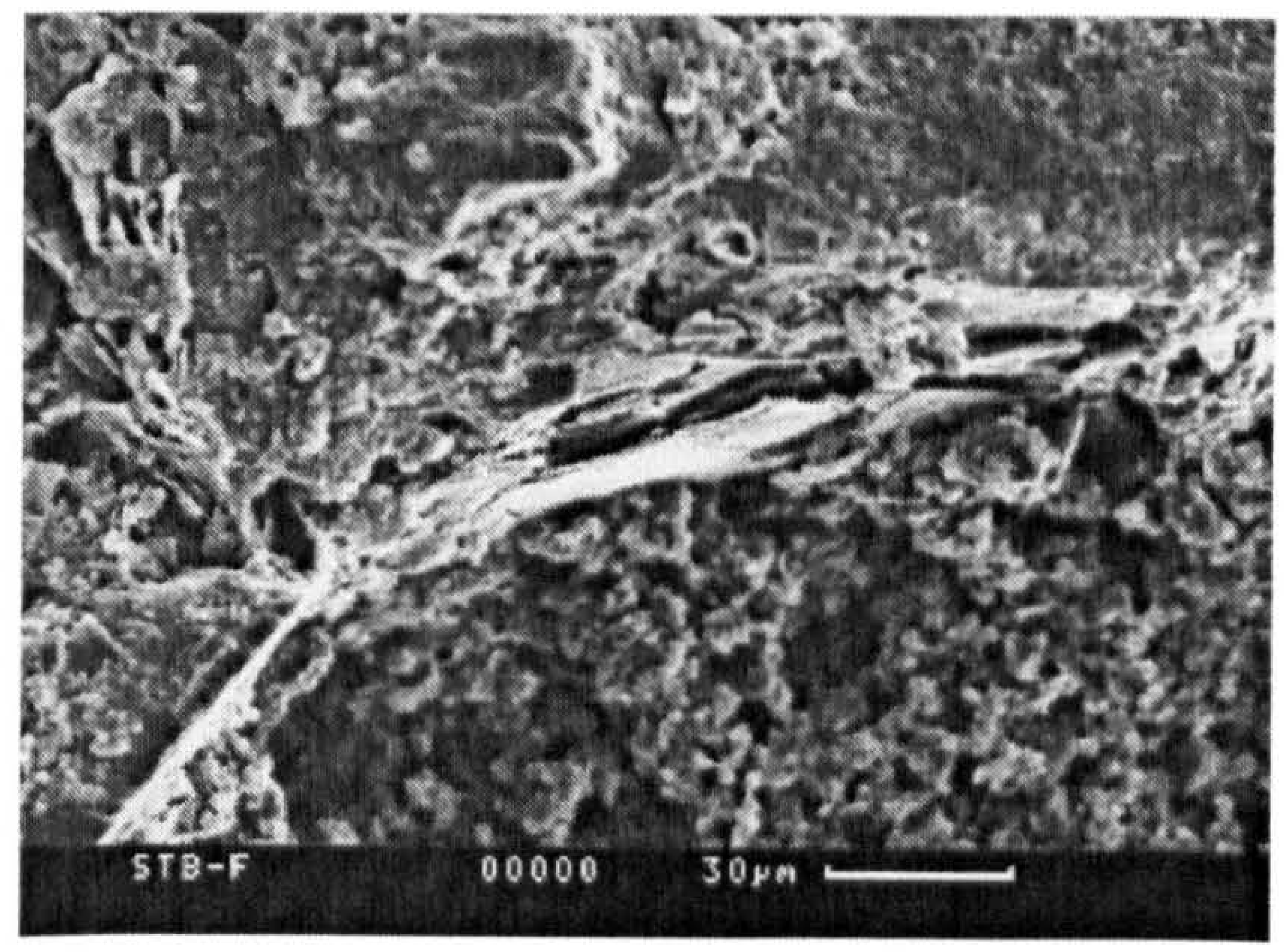
(a) Micaceous sandstone (30μm)



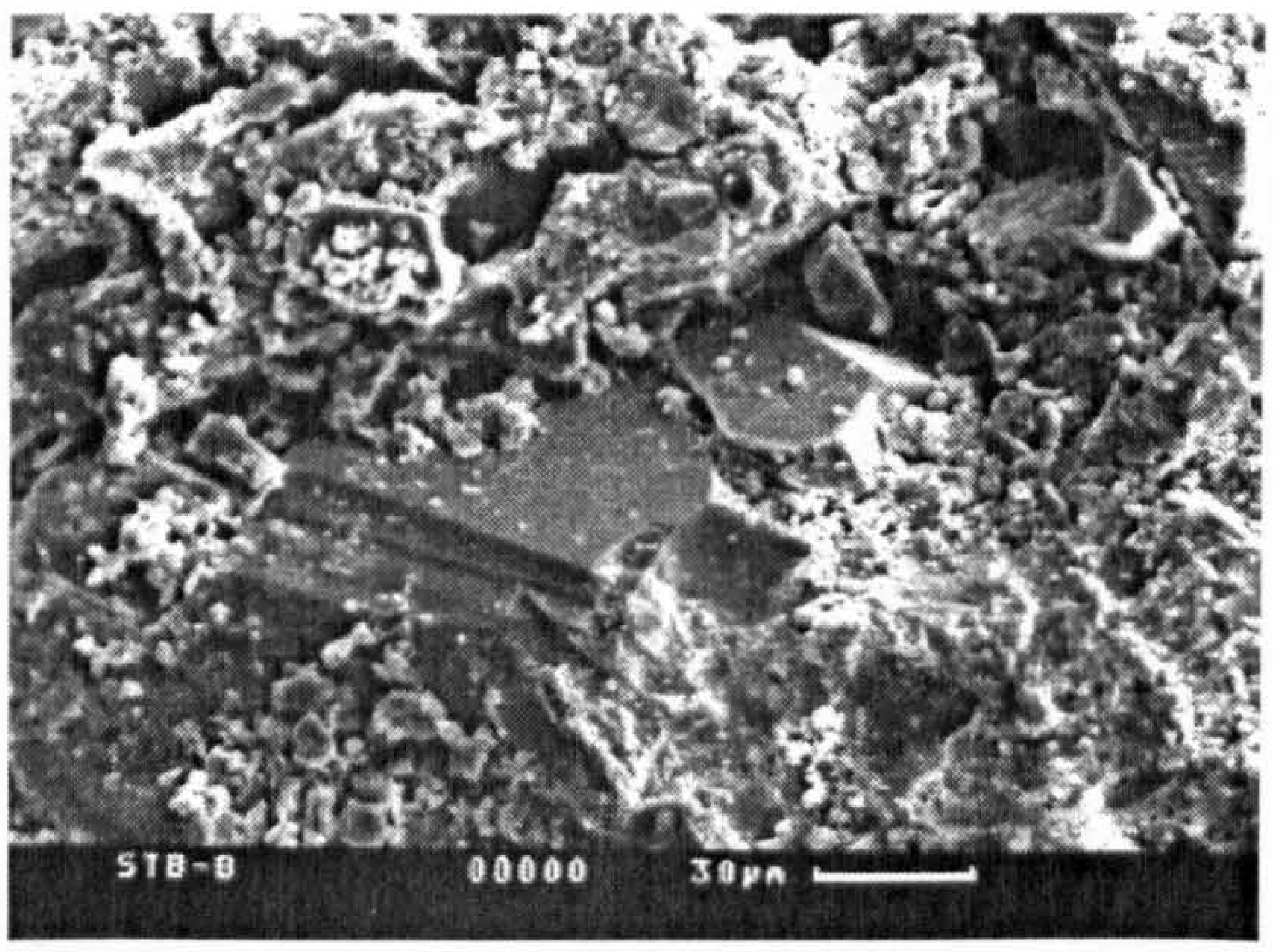
(b) Micaceous sandstone (30μm)



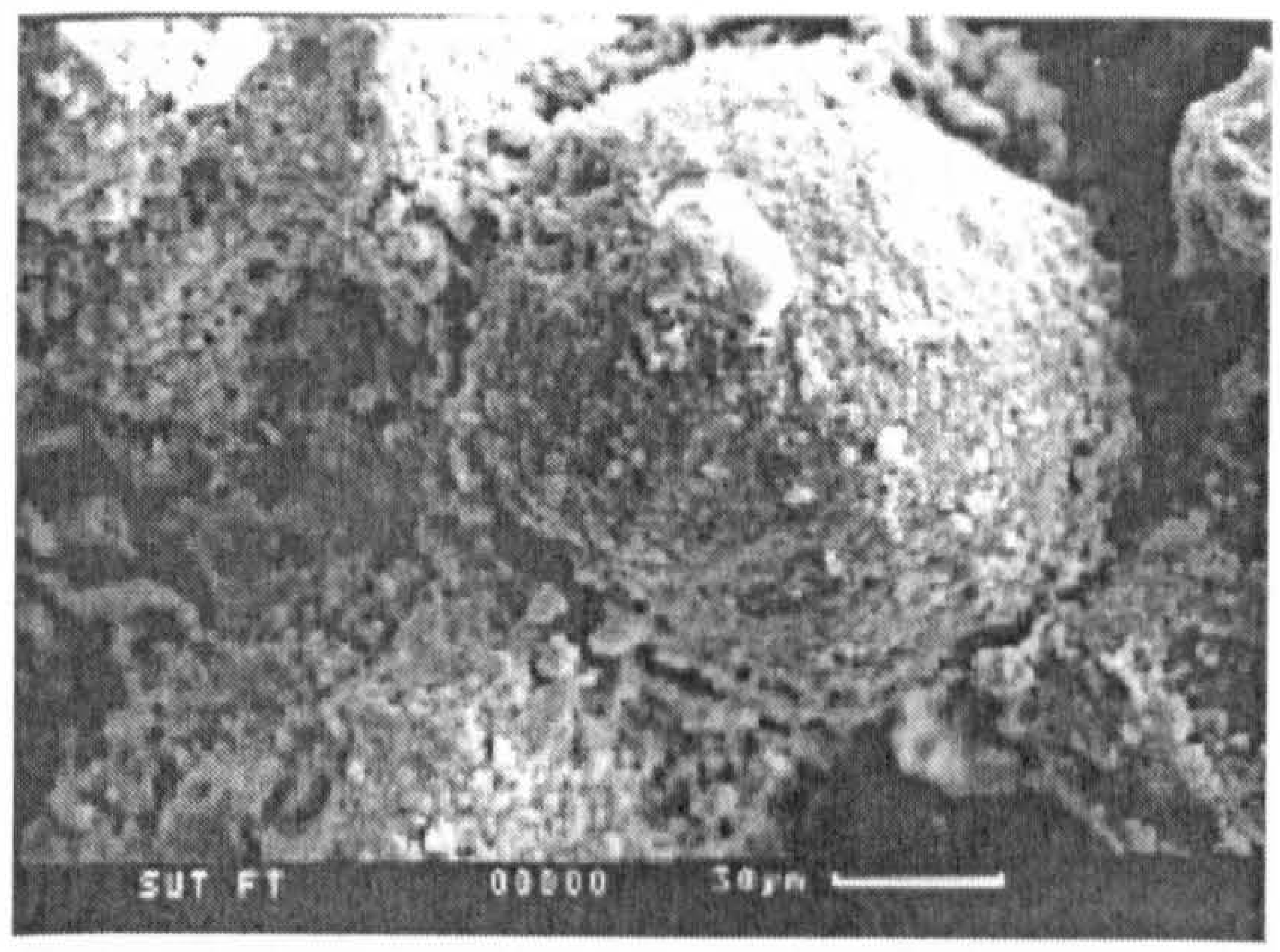
(c) Micaceous sandstone (10μm)



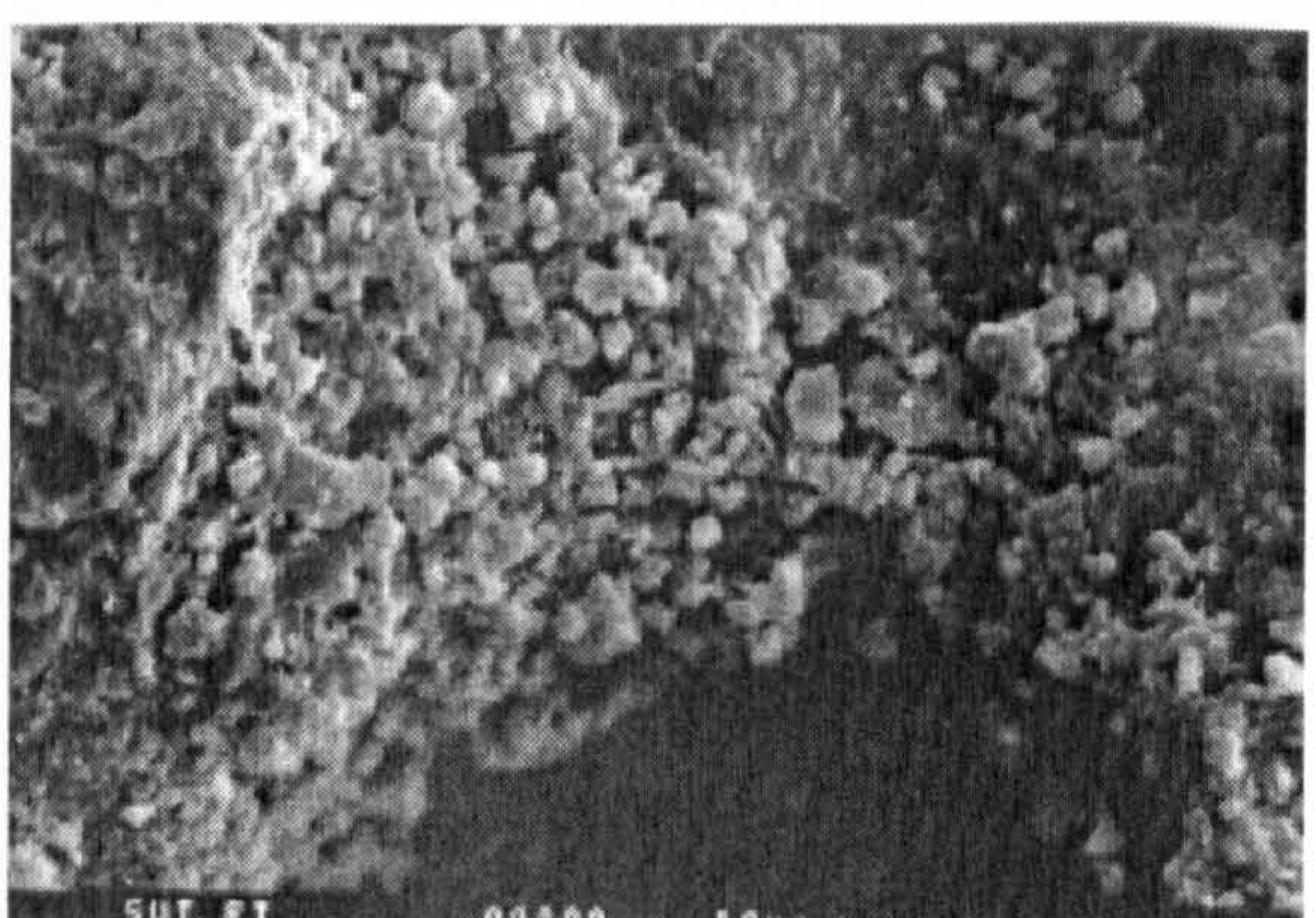
(d) Micaceous sandstone (30μm)



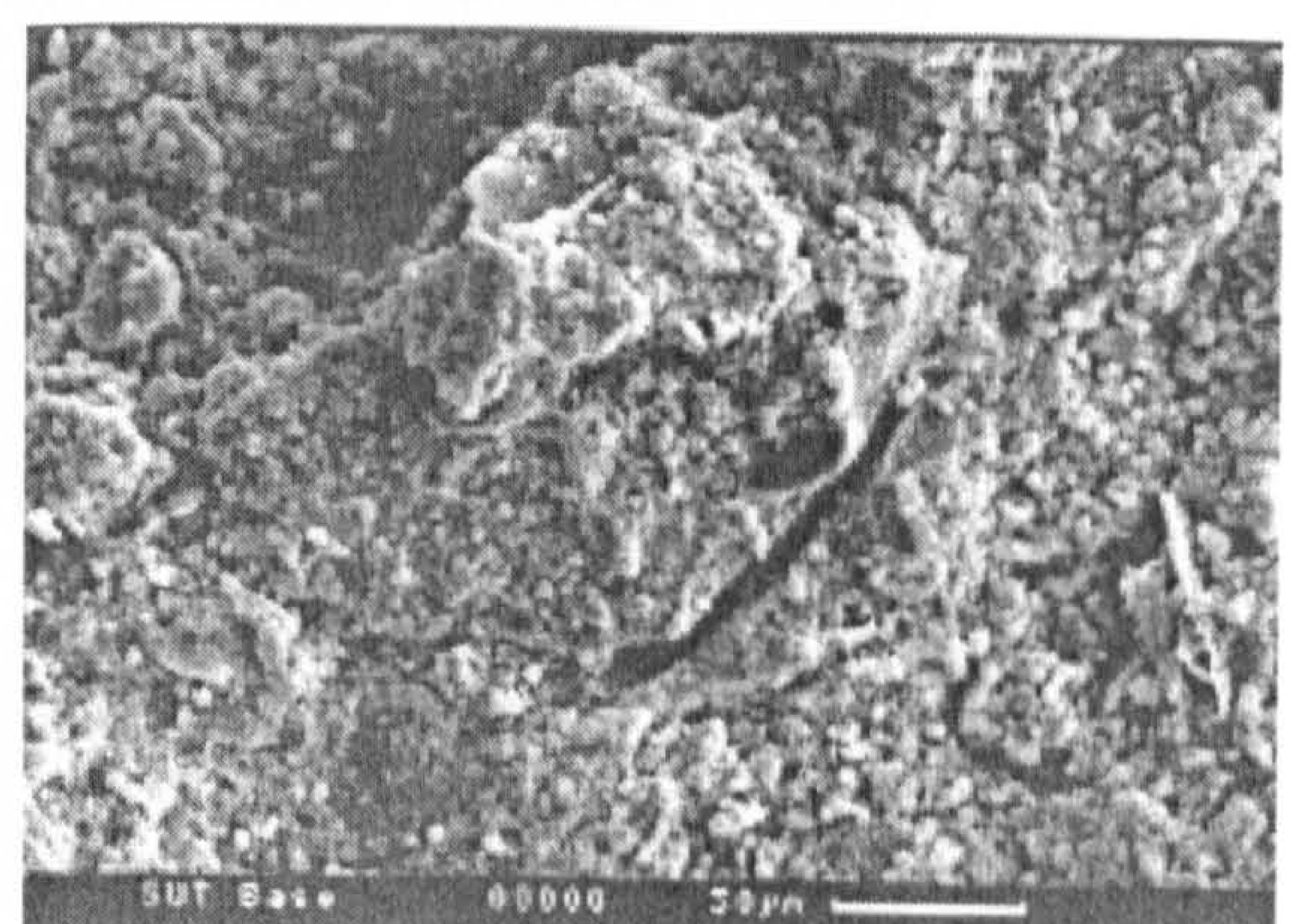
(e) Micaceous sandstone (30μm)



(f) Calcareous sandstone (30μm)

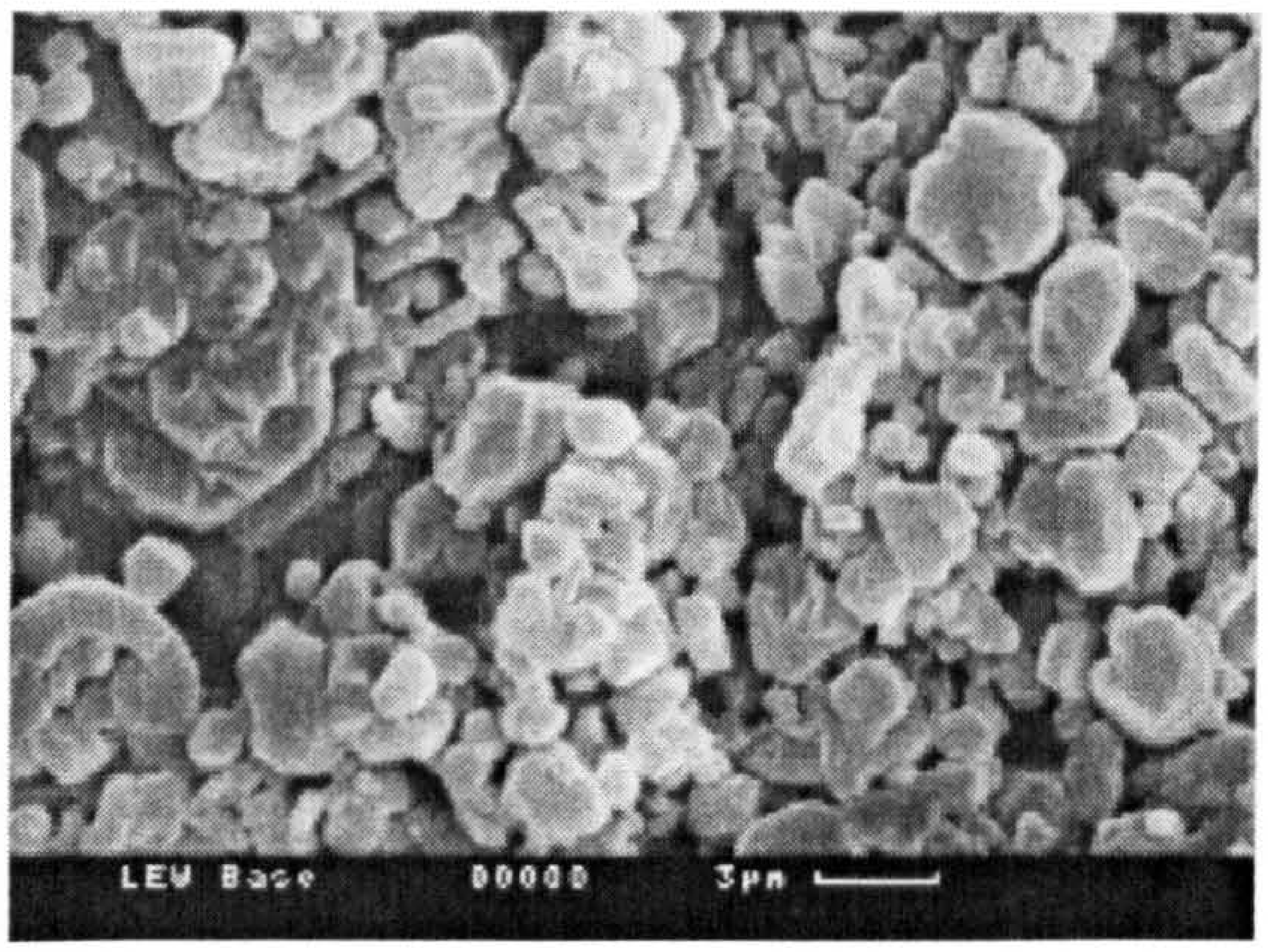


(g) Calcareous sandstone (10μm)

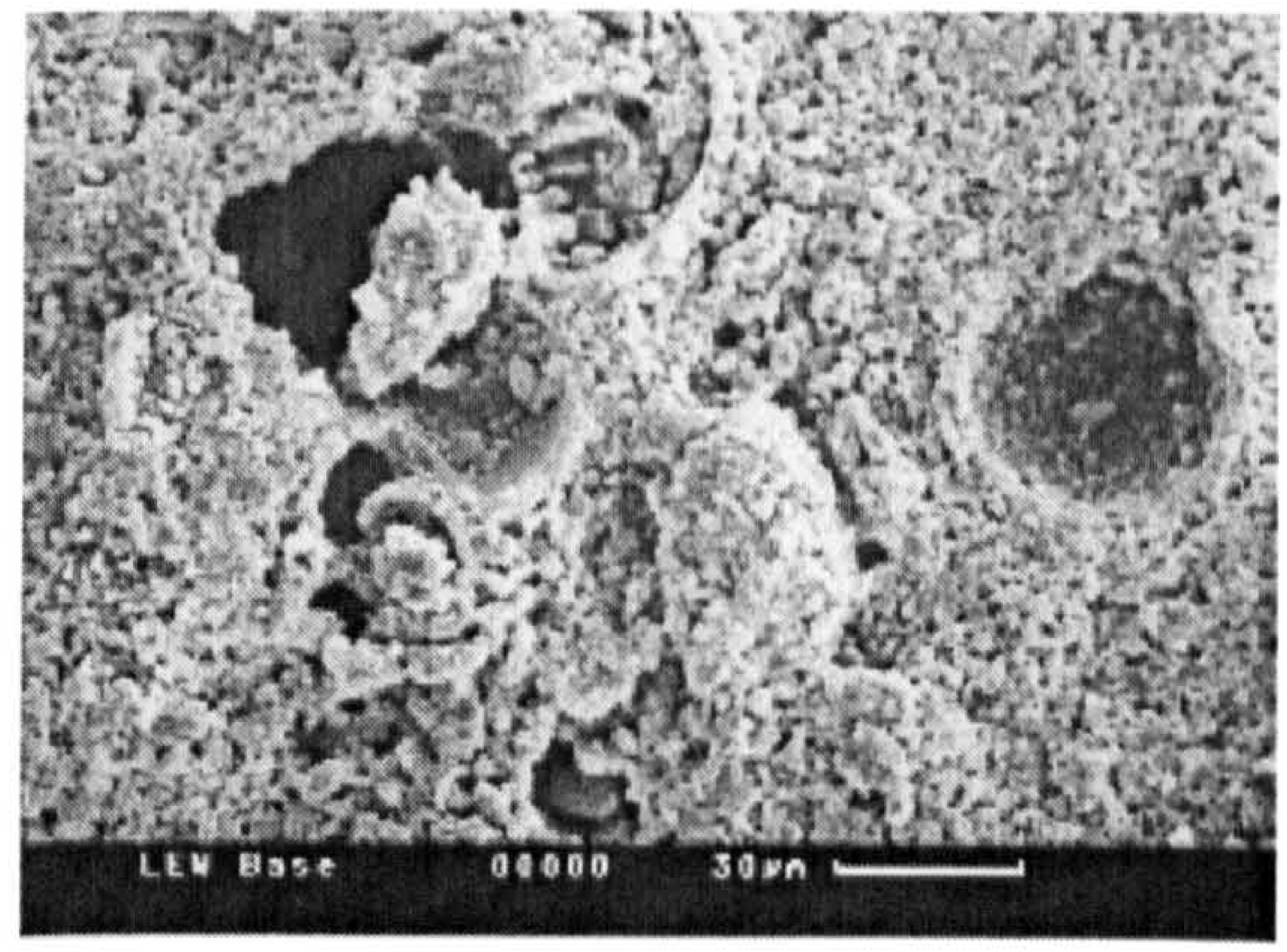


(h) Calcareous sandstone (30μm)

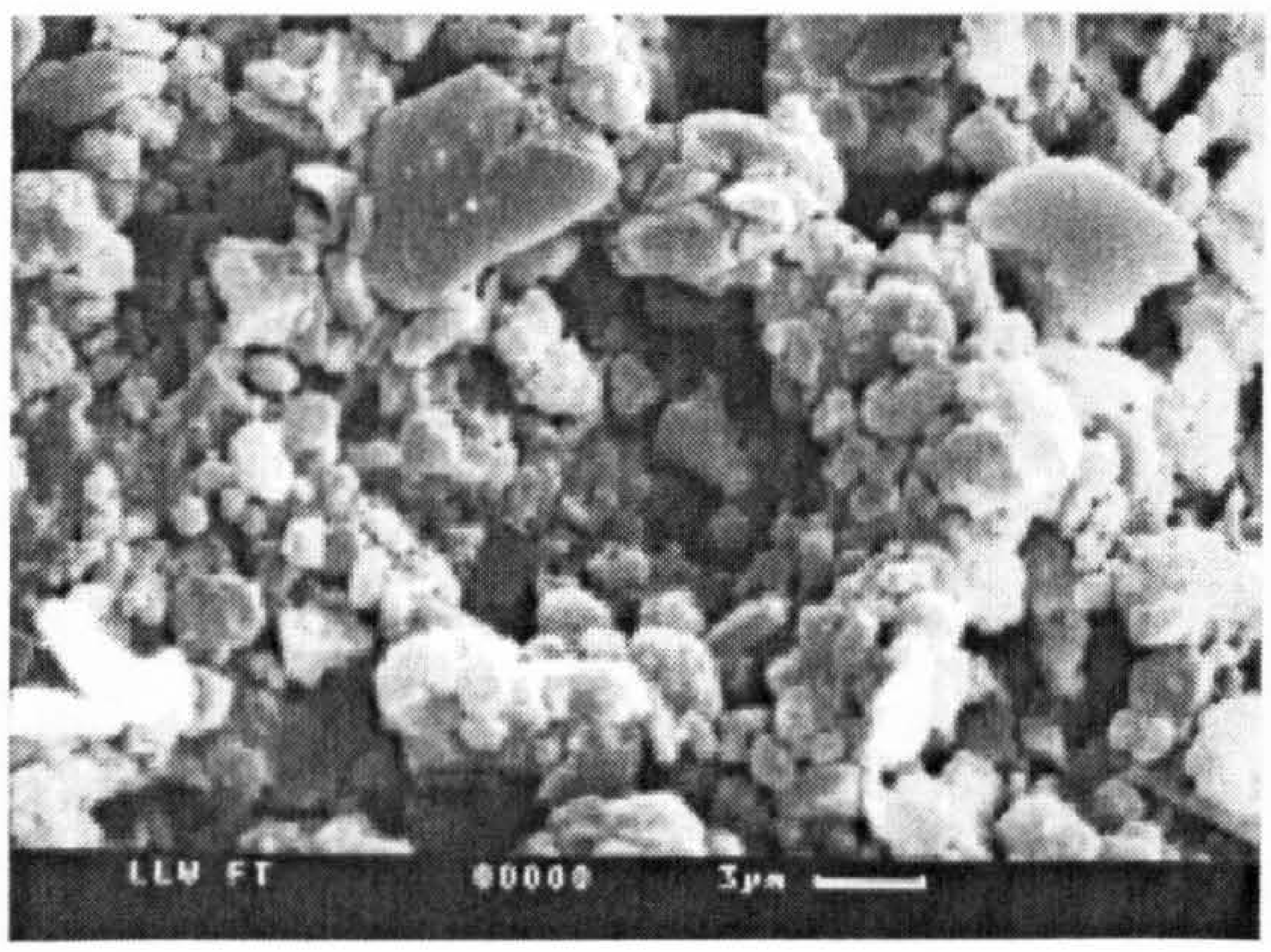
Plate 4.3 Pre-test scanning electron micrographs
Refer to text for explanation (numbers in parenthesis give length of scale bar)



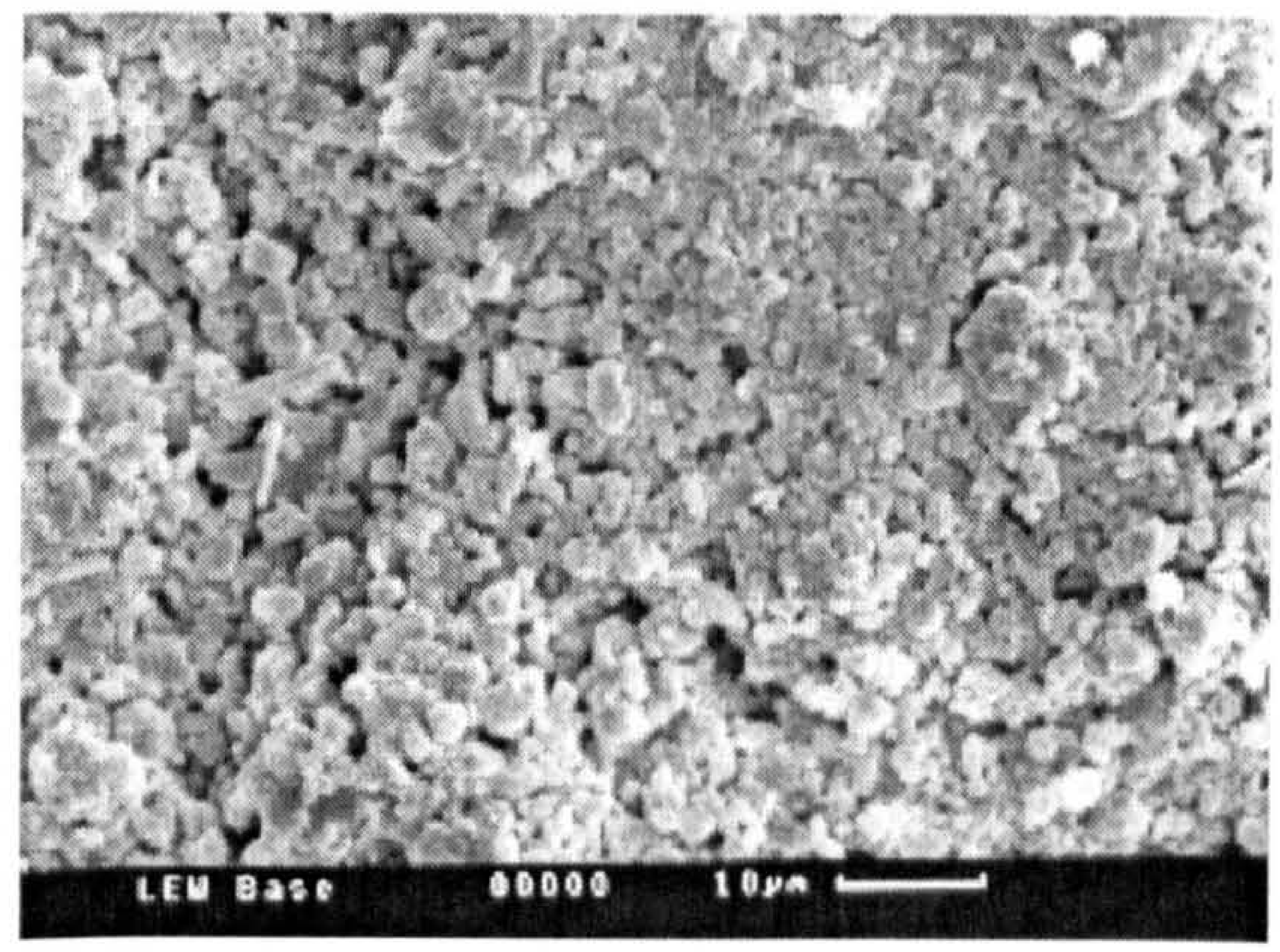
(a) Low density chalk (3μm)



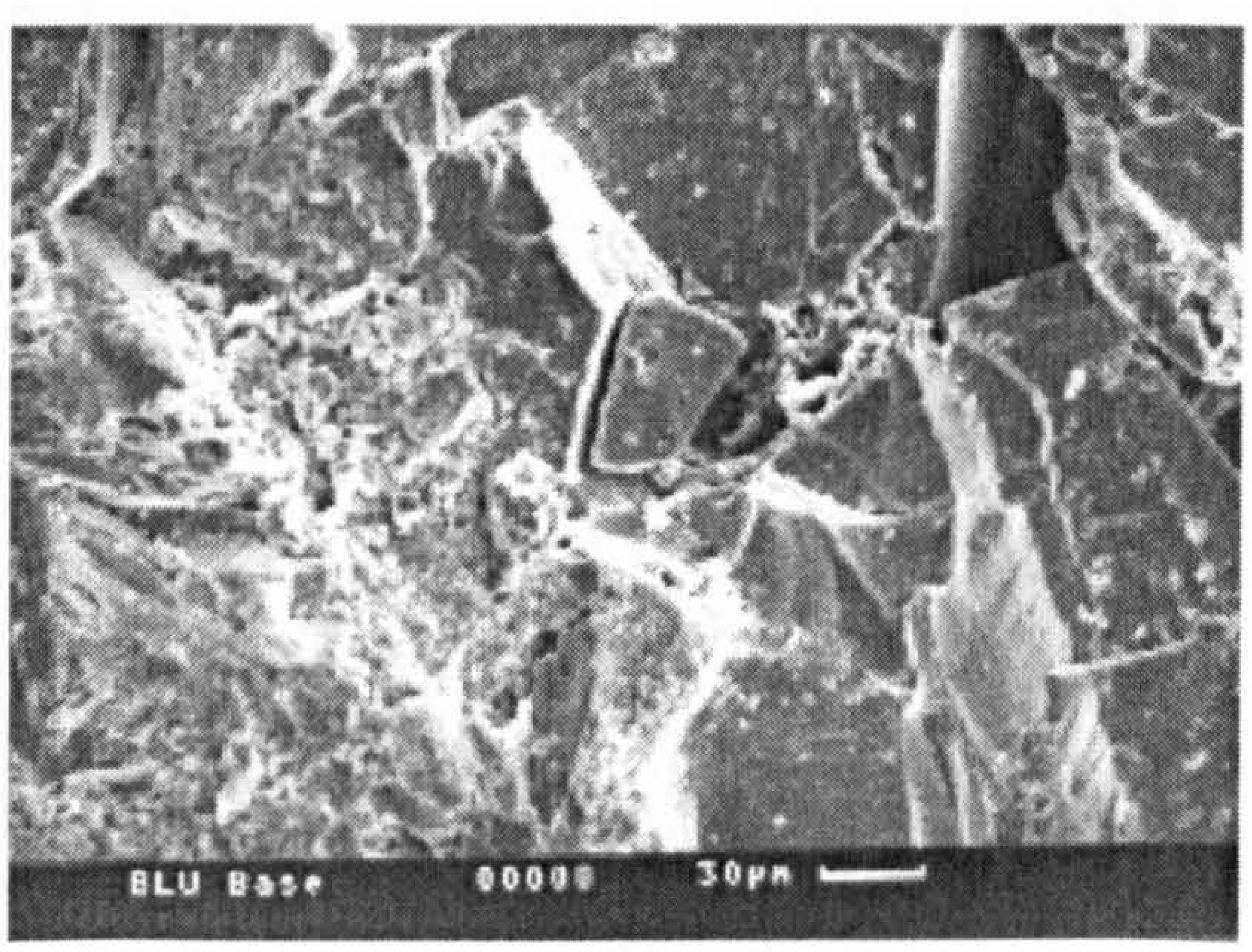
(b) Low density chalk (30μm)



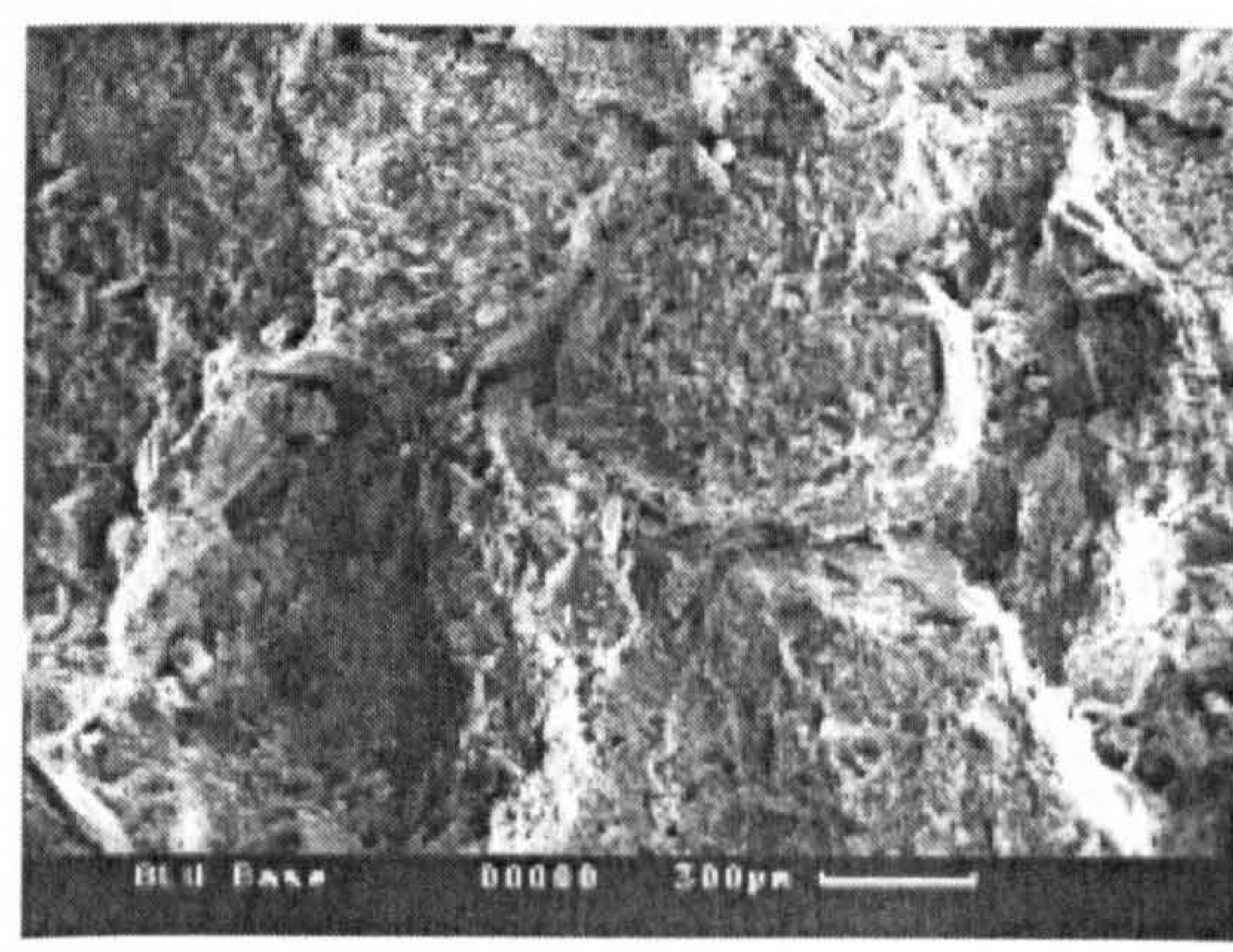
(c) Low density chalk (3μm)



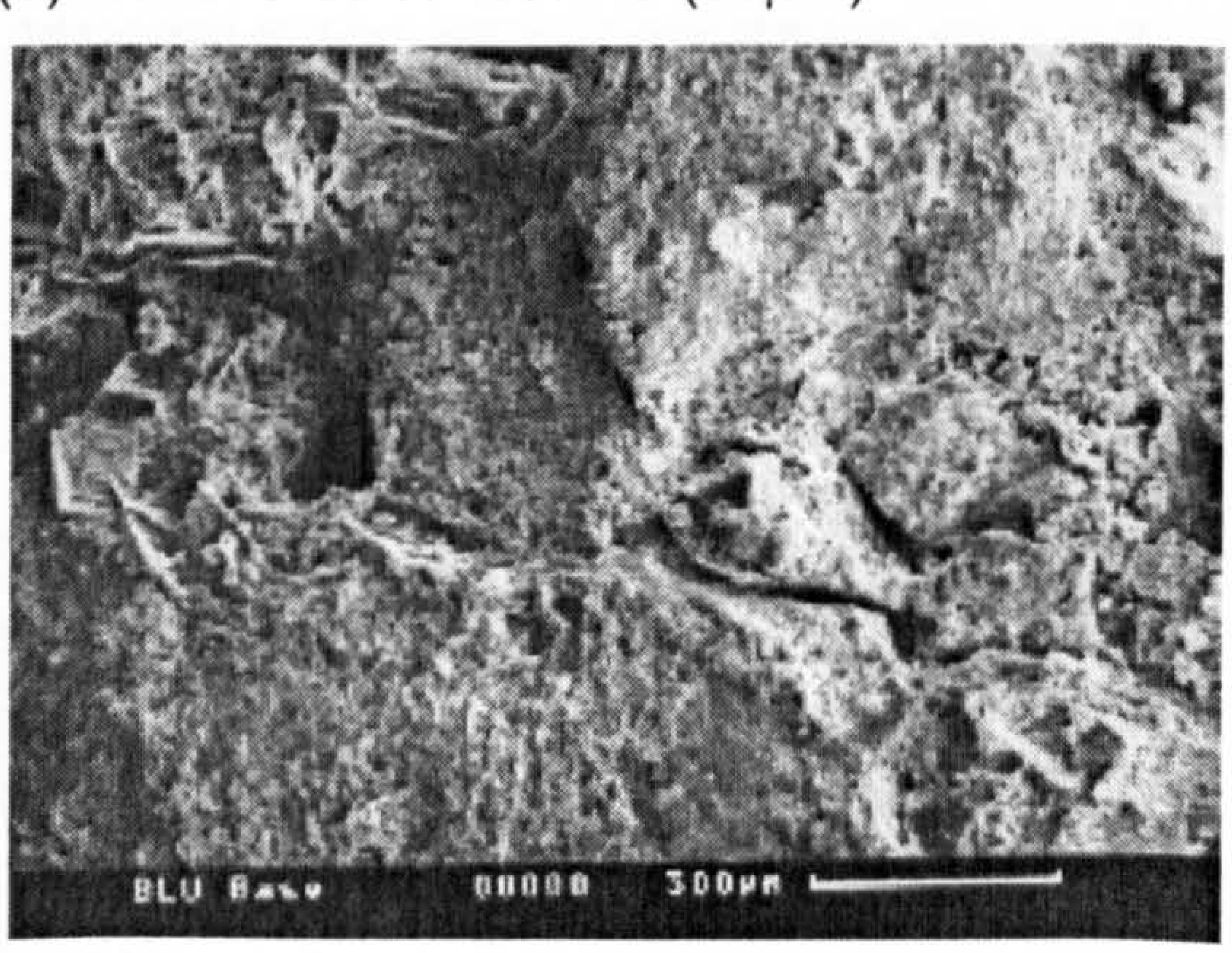
(d) Low density chalk (10μm)



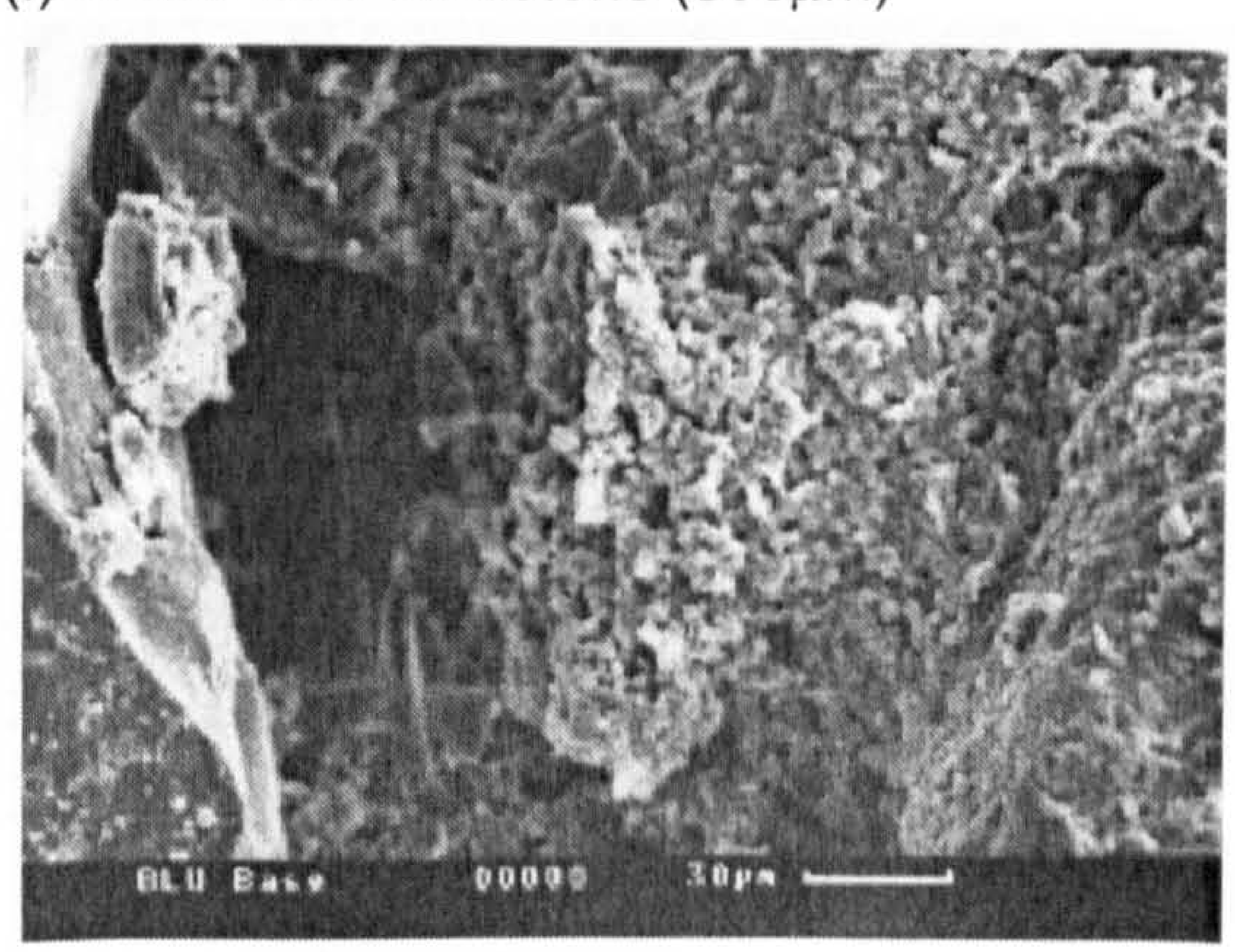
(e) Weathered sandstone (30μm)



(f) Weathered sandstone (300μm)

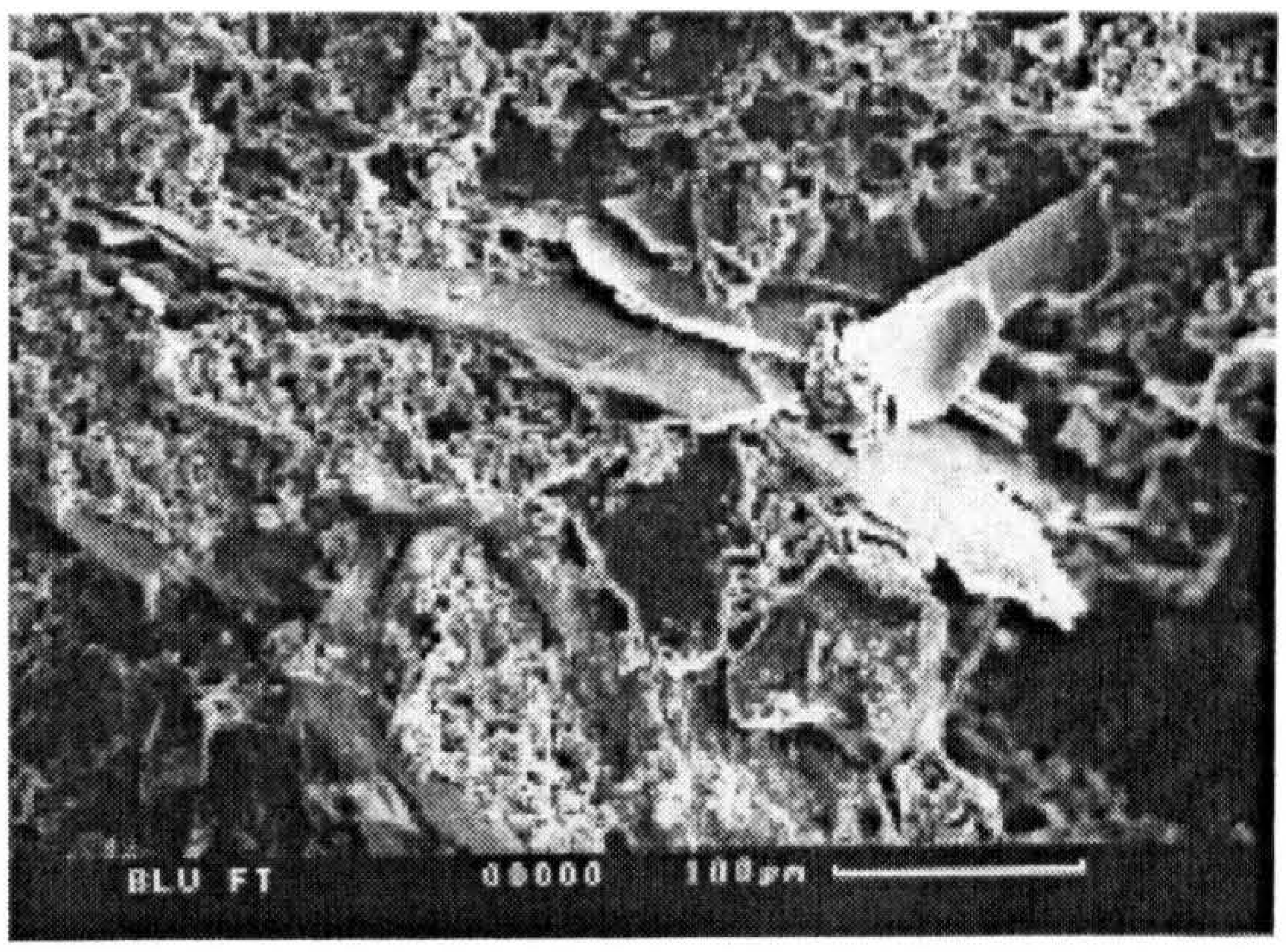


(g) Weathered sandstone (300μm)

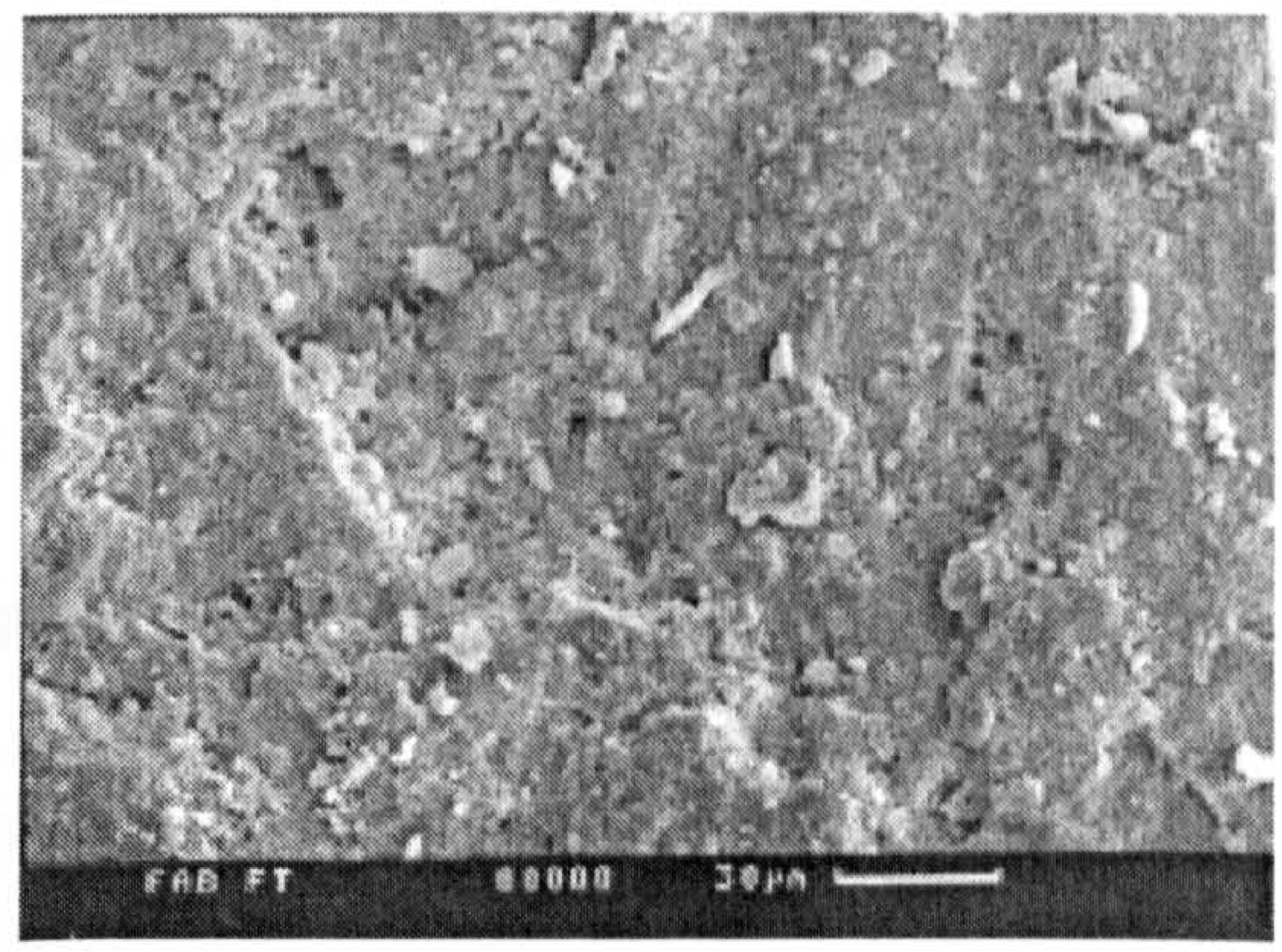


(h) Weathered sandstone (30μm)

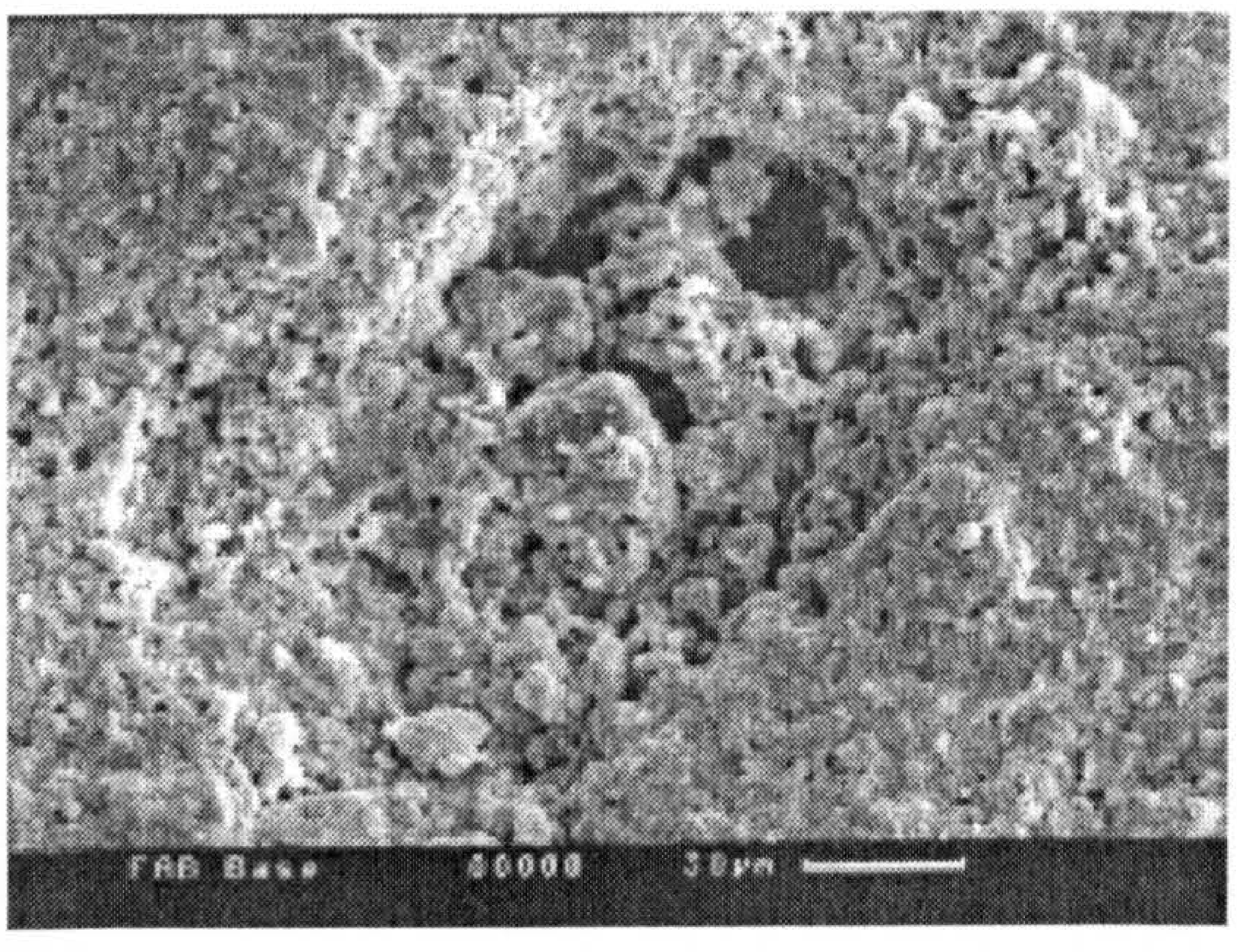
Plate 4.4 Pre-test scanning electron micrographs
Refer to text for explanation (numbers in parenthesis give length of scale bar)



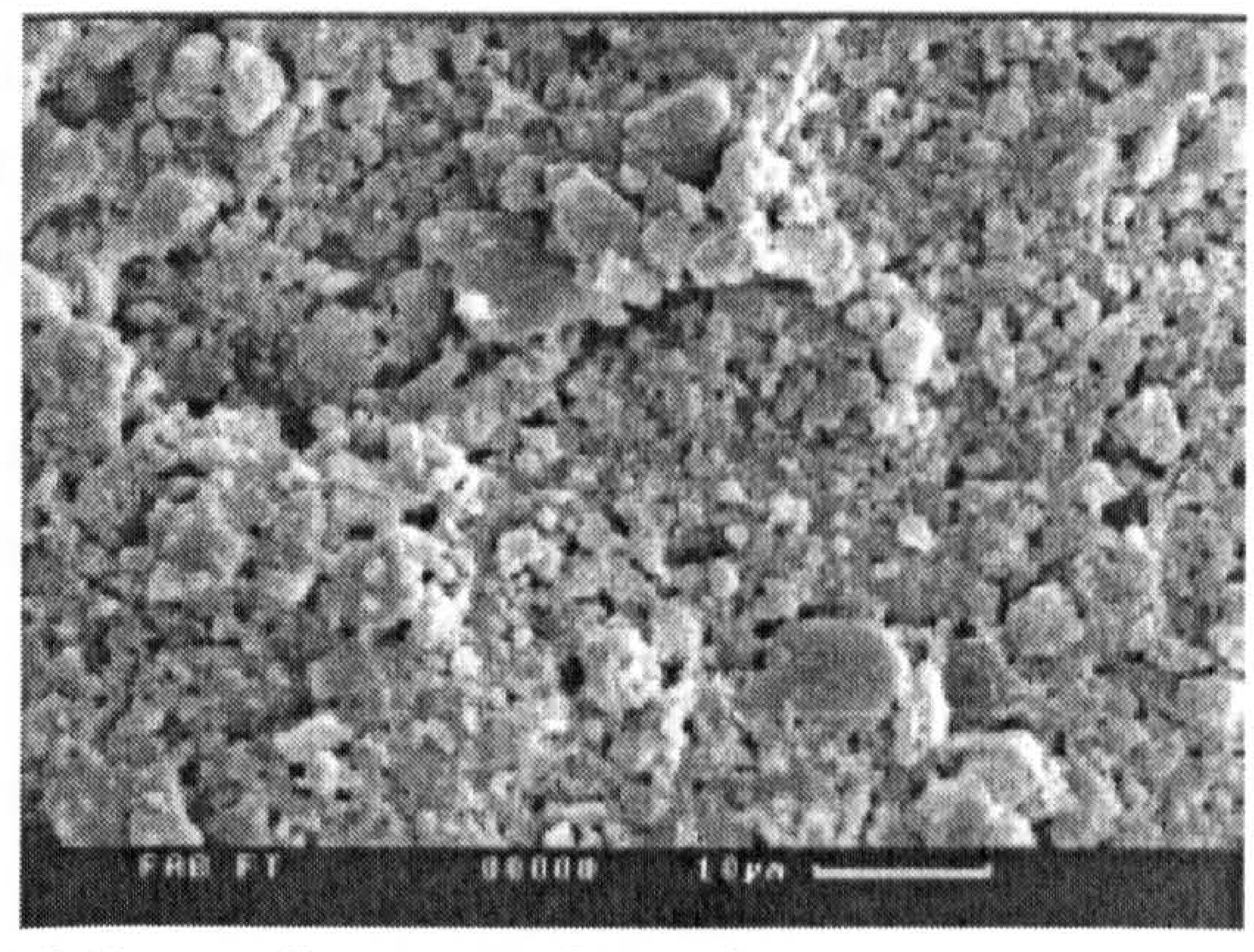
(a) Weathered sandstone (100μm)



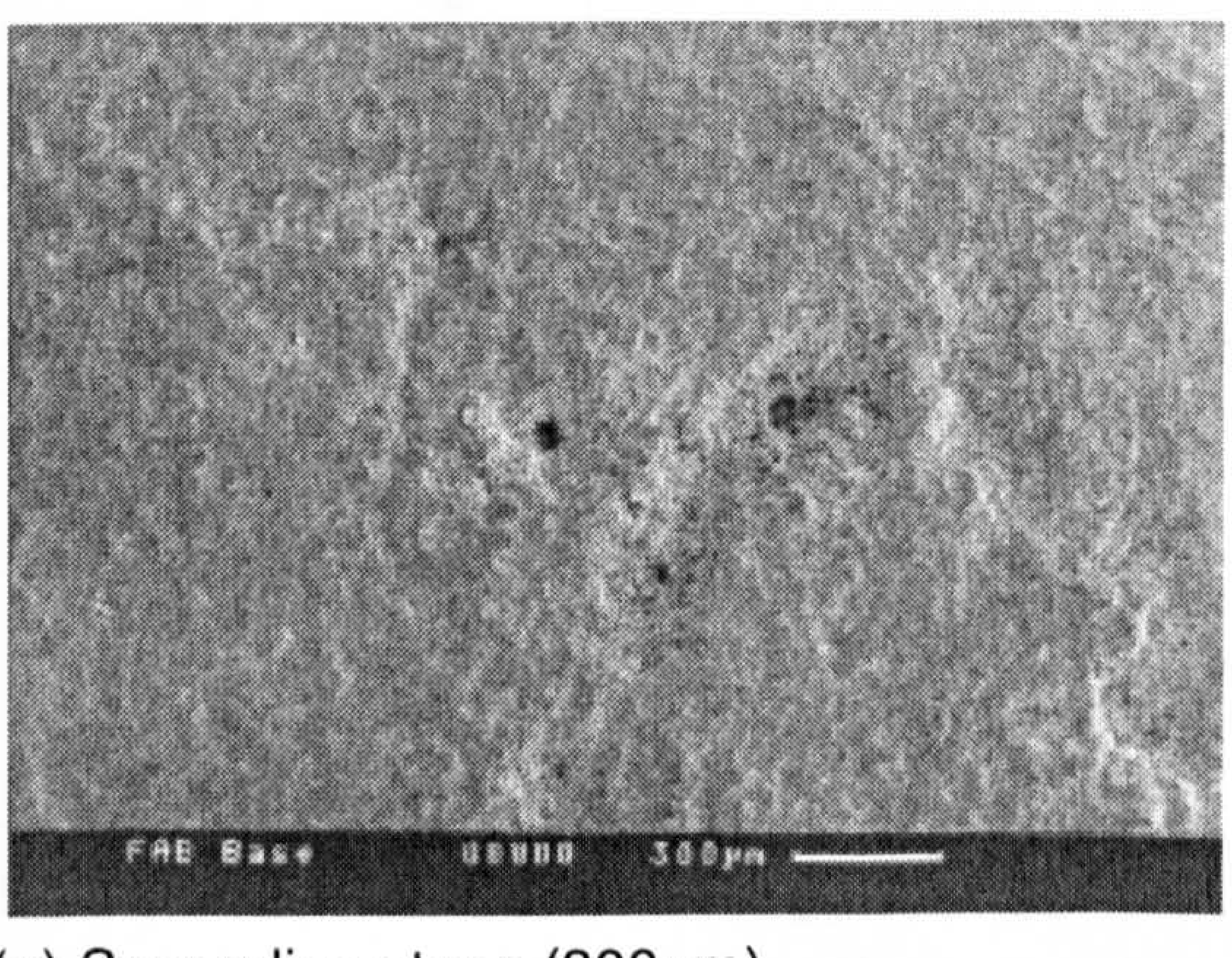
(b) Sparry limestone (30μm)



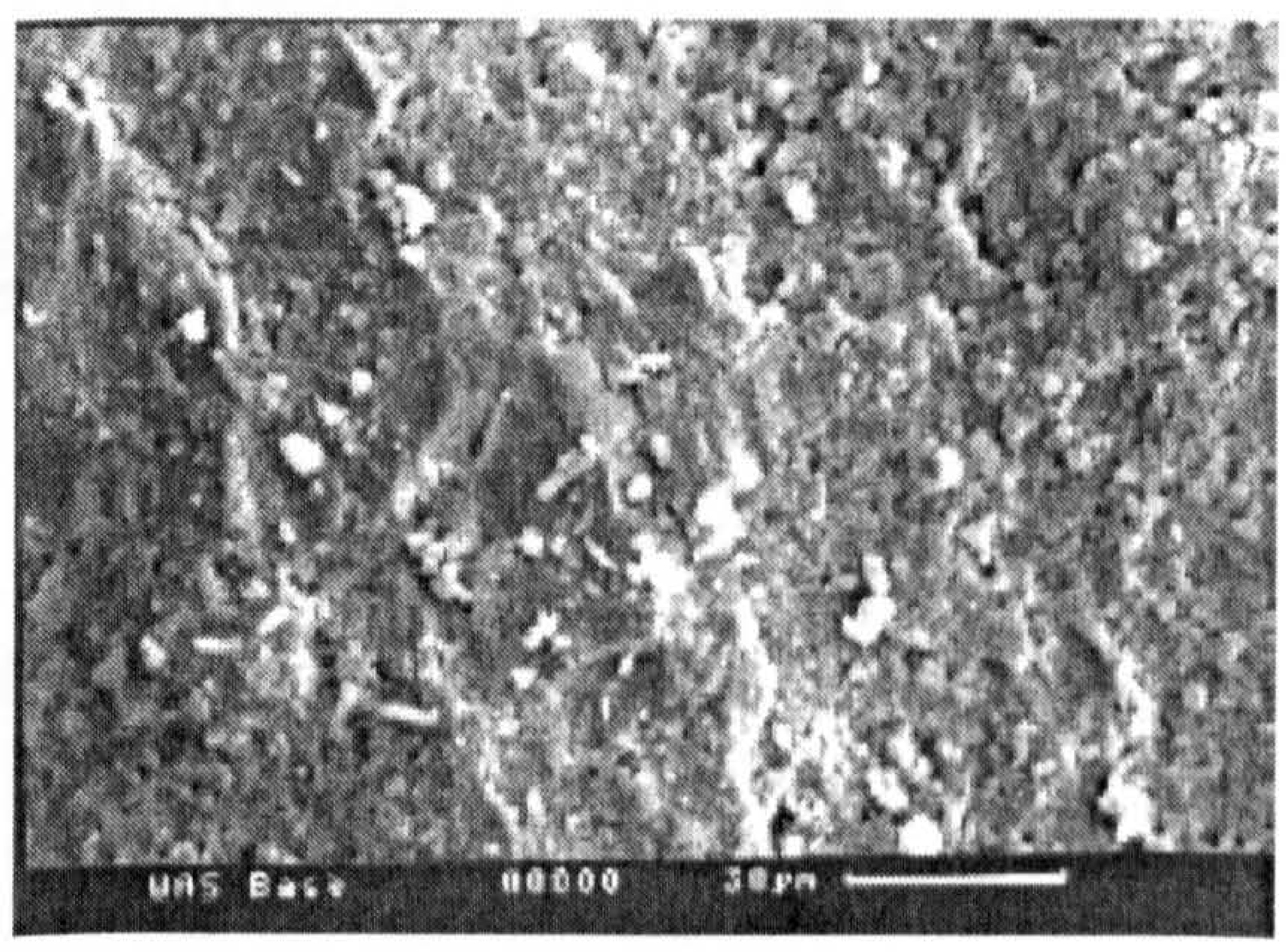
(c) Sparry limestone (30μm)



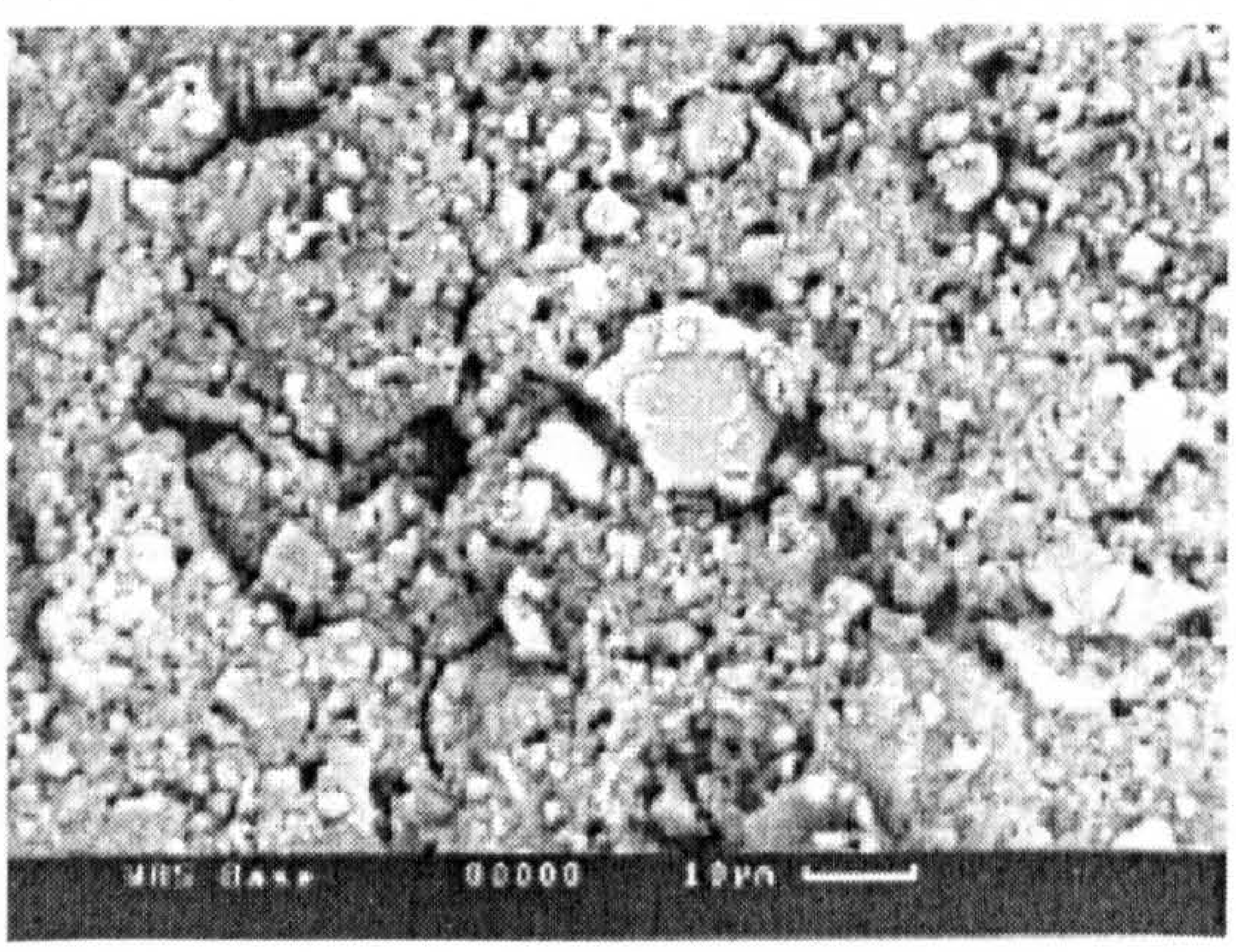
(d) Sparry limestone (10μm)



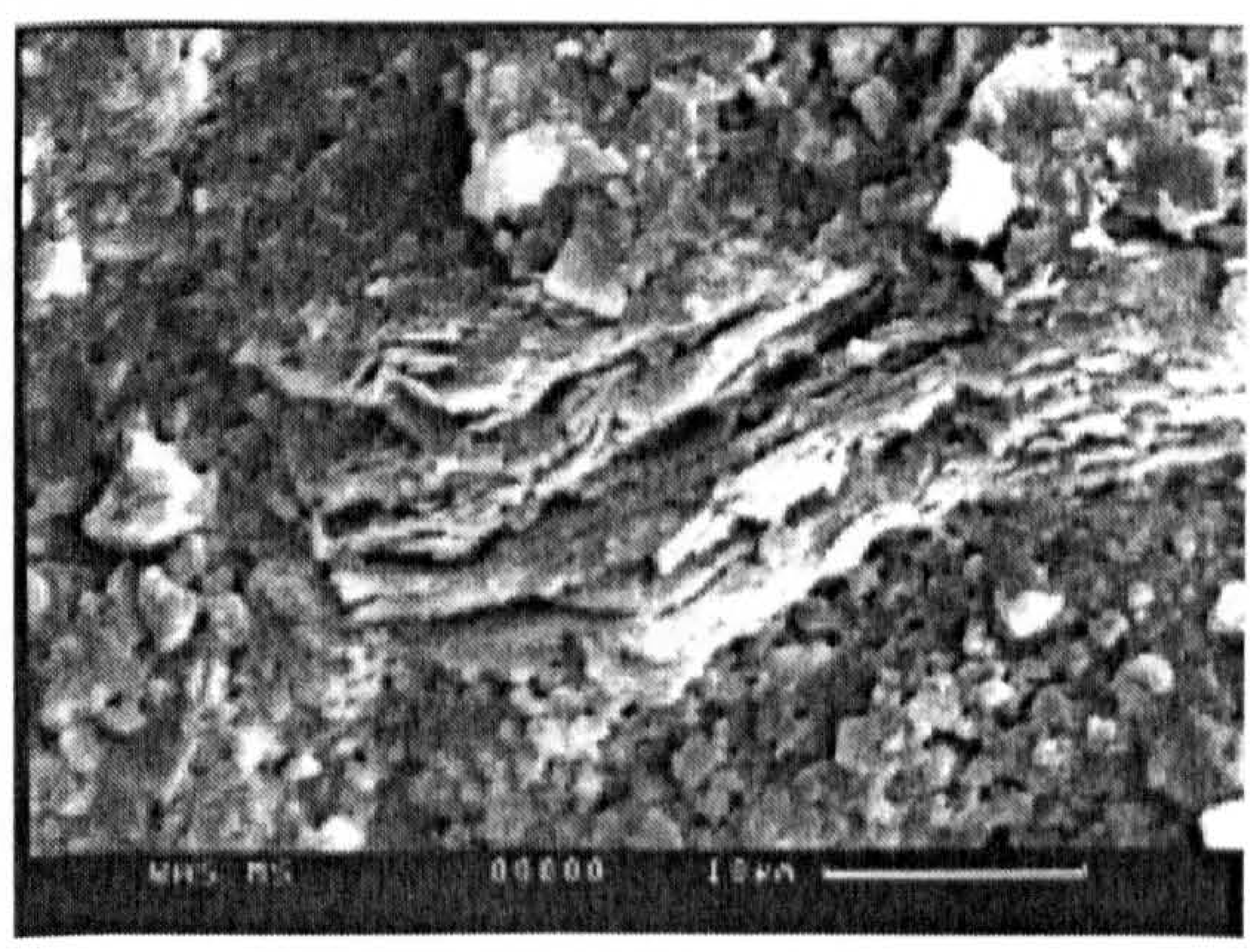
(e) Sparry limestone (300μm)



(f) Metasediment (30μm)



(g) Metasediment (30μm)



(h) Metasediment (10μm)

Plate 4.5 Pre-test scanning electron micrographs
Refer to text for explanation (numbers in parenthesis give length of scale bar)

There is no evidence from SEM analysis which corroborates the large peak pore diameters indicated by mercury intrusion, either for SpaL or MetS. It is therefore considered prudent to treat these data with extreme caution.

4.5 Mechanical Properties

4.5.1 Compressive strength C_o, Modulus of Rupture T_{mr}, and Point Load Strength IS_{50}

Some mechanical and lithological properties of the rocks tested here are given in Table 4.9. There is a very close correlation between C_o, T_{mr} and IS_{50} as indicated in Figure 4.18. There are several published values for the ratio of uniaxial compressive strength to point load strength. Examples are quoted in Bowden et al (1998) for a wide range of sedimentary rocks varying from 3 to 27. Brook (1993) quotes value of 7 to 25 for both sedimentary and igneous rocks. For the rocks tested here the ratio varies from 9 to 18 with values of 12 to 15 being the most common. Tensile strength (T_o) is often about $0.1C_o$ (Brook 1993), but since $0.5T_{mr} \sim T_o$ (Brook 1990), an expected ratio between C_o and T_{mr} would be 5. In practice, for the rock tested here, this ratio varies from 2.4 to 6.6, with the largest deviations from 5 occurring in the weakest rocks.

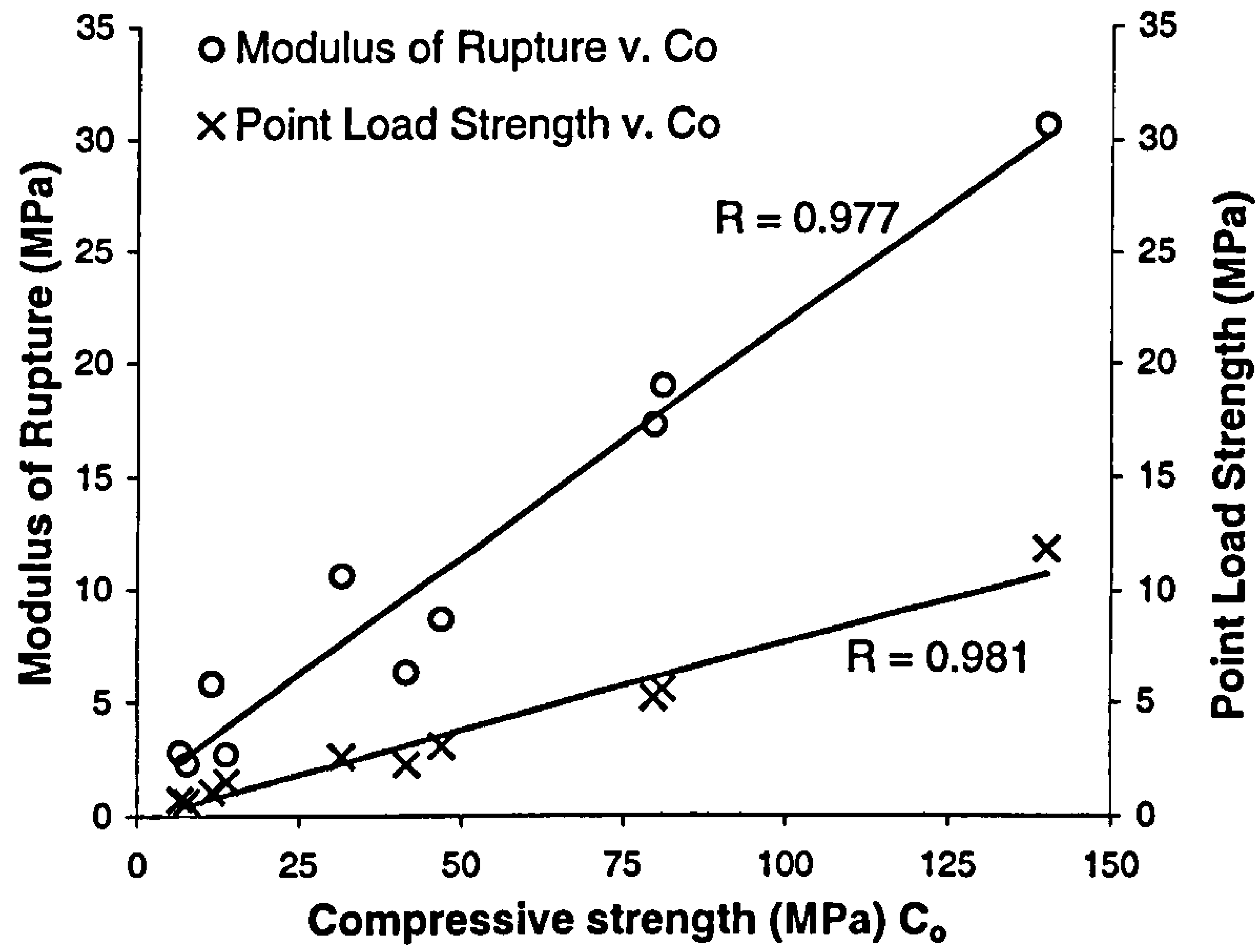


Figure 4.18 Correlation between mean pre-test mechanical rock properties for the ten rock types

A wide range of compressive strengths is represented in the rocks tested, varying from 7 to 140MPa. Difficulties were encountered in coring the metasediment and so it was not possible directly to determine a compressive strength value. As an alternative, an estimate was obtained based on the mean of two ratios, C_o/T_{mr} (4.1) and C_o/IS_{50} (13.1) for the other rock types, giving a range of 125 to 155MPa for MetS. A mean value of 140MPa is used.

4.5.2 Sonic velocity and dynamic modulus of elasticity (E_{dyn})

There is a wide range of P-wave propagation velocities, varying from 1785 to 6239ms^{-1} for the rocks. Since wave propagation velocity is reduced by the presence of voids, it is reasonable to expect values to correlate well with other rock properties such as density, porosity and mechanical strength. In fact there are good linear relations between these properties as indicated by correlation coefficients of 0.73 to 0.79 for the three strength parameters, 0.78 for dry density and –0.68 for porosity. These are based on correlation of mean pre-test values for all rocks. It is to be expected that the relationship between Vp and n_e is weaker, since while the latter represents interconnected void space, the former should also reflect unconnected void space.

Dearman et al (1987) established good relations between sonic velocity and the percentage of sound mineral constituents. All of the velocities recorded here, with the exception of SpaL, are significantly less than if calculated on the basis of the velocity of individual mineral constituents. This is largely a reflection of the amount of void space contained within, in the form of pores, microcracks and macrofractures (Goodman 1989).

Young's dynamic modulus of elasticity varies from 5.2 to 35.0 and correlates well with density and the three strength parameters (r = 0.85 to 0.86).

ROCK TYPE	C_o	T_{mr}	IS_{50}	V_p	V_s	E_{dyn}
LdCh	6.64 (1.89)*1	2.8 (0.63)	0.75 (0.13)	1966 (1451)	1911 (741)	9.44 (1.51)
MagL	7.77 (1.72)	2.3 (0.57)	0.62 (0.19)	2134 (71)	1104 (48)	5.23 (0.39)
OolL	11.56 (5.6)	5.81 (0.88)	1.00 (0.50)	3616 (409)	1526 (125)	14.04 (2.05)
HdCh	46.95 (5.46)	8.67 (1.05)	3.07 (0.51)	3544 (98)	1678 (32)	15.33 (0.70)
SpaL	79.72 (23.42)	17.26 (5.36)	5.19 (0.74)	6239 (404)	2139 (124)	35.03 (4.02)
WeaS	13.79 (2.42)	2.71 (0.74)	1.52 (0.61)	1785 (90)	1080 (47)	6.30 (0.59)
CalS	31.55 (6.06)	10.60 (2.05)	2.63 (0.59)	3284 (215)	1512 (33)	12.17 (0.60)
MicS	41.55 (19.18)	6.28 (1.37)	2.26 (0.37)	3006 (104)	1494 (44)	12.49 (0.89)
LamZ	81.03 (12.42)	18.98 (2.57)	5.56 (1.55)	3476 (243)	1885 (94)	23.08 (2.17)
MetS	140*2	30.65 (13.02)	11.82 (1.96)	5333 (487)	1918 (358)	28.54 (8.69)

Table 4.9 Summary of pre-test sample mean data for mechanical properties*3
Where C_o = compressive strength MPa; T_{mr} = modulus of rupture MPa;
IS_{50} = point load strength MPa; V_p = P-wave ultrasonic pulse velocity ms^{-1};
V_s = S-wave ultrasonic pulse velocity ms^{-1}; E_{dyn} = dynamic modulus of elasticity GPa.

Note 1 Numbers in parentheses are standard deviations
Note 2 An estimate based on C_o/T_{mr} and C_o/IS_{50} for all samples
Note 3 The number of specimens upon which each mean is based is given in Table 3.3

4.6 Correlation of Rock Properties with Deterioration

In this next section, the objective is to explore any statistical correlation between a number of rock properties and deterioration as determined from weight loss, fracture density and index of fracture porosity. Emphasis is placed on those properties where some correlation with deterioration might be expected. From discussions above, effective porosity, saturation coefficient, microporosity, point load strength and the dynamic modulus of elasticity have been selected for evaluation. Some caution is needed in the interpretation of data requiring destructive testing (microporosity and point load strength) since measurements are based on test specimens representative of the sample rather than the actual specimens used in the weathering tests. Also, note that the other data (effective porosity, saturation coefficient and elasticity) are mean values for each sample and inevitably do not reflect the range of values often observed for individual specimens.

4.6.1 Freeze-thaw

Because of the close correlation between weight loss and fracture density for the freeze-thaw test (Figure 4.12) it should be assumed that comments made for weight loss also apply to fracture density unless indicated otherwise.

From figures 4.19a and 4.20a, a general trend for increasing weight loss with higher effective porosity is indicated. OolL is a notable anomaly here, suffering considerably more breakdown than other rocks with a similar n_e. This is clear indication that factors other than pore volume influence susceptibility to freeze-thaw and in this case it might relate to the weakness of grain cementing material. LamZ is also anomalous in that a very high fracture density was recorded despite a relatively low porosity. In this case, the nature of breakdown and the susceptibility of the rock is controlled by the structural weakness of closely spaced laminations.

There is no clear relationship between weight loss and saturation coefficient (Figure 4.19b) even if the spurious data points for SpaL and MetS and the anomalous point MagL are removed (see earlier discussion in section 4.4.5). However, when considered in relation to fracture density, and removing the same doubtful data points, there appears to be a very strong correlation with S (Figure 4.20b). There is also a suggestion in both sets of data that a substantial increase in susceptibility occurs where S exceeds 0.85.

Although points are scattered, particularly for the weight loss data, there is a general positive correlation between deterioration and microporosity (Figure 4.19c and 4.20c). This is to be expected given the positive correlation between S and μn_m as discussed earlier.

Broad inverse correlation between weight loss and both point load strength and elasticity is detectable (Figure 19d and 19e), particularly among the calcareous rocks, but there are some notable exceptions. In particular, the three sandstones (WeaS, CalS, MicS) and MagL show a distinctive resilience to freeze-thaw despite their relatively low strength and elasticity. In contrast, OolL has a similar elasticity to several other rocks in which considerably less breakdown occurred (HdCh, CalS, MicS). Correlation of fracture density with the same rock

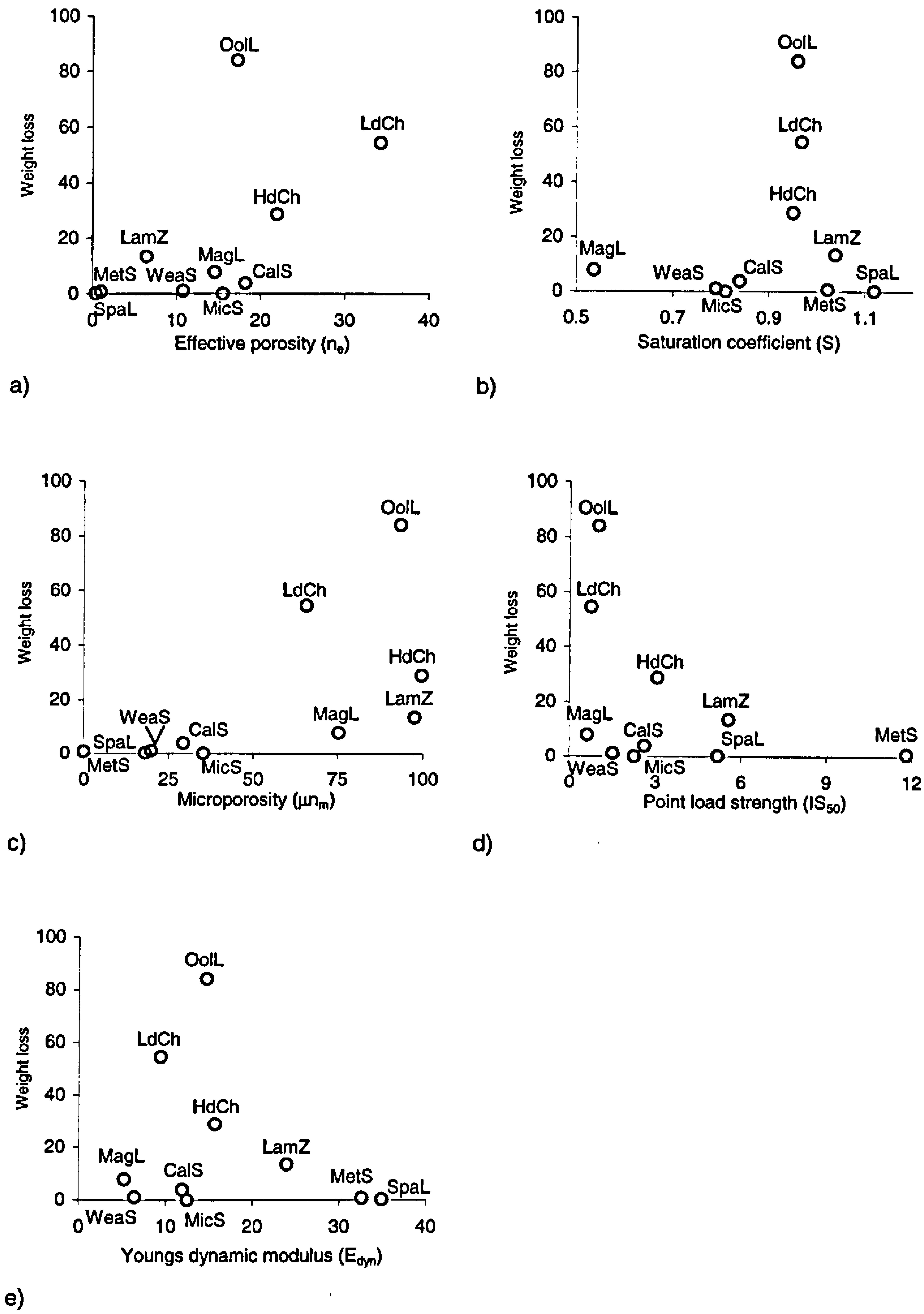


Figure 4.19 (a-e) Correlation of rock properties with weight loss for the freeze-thaw test

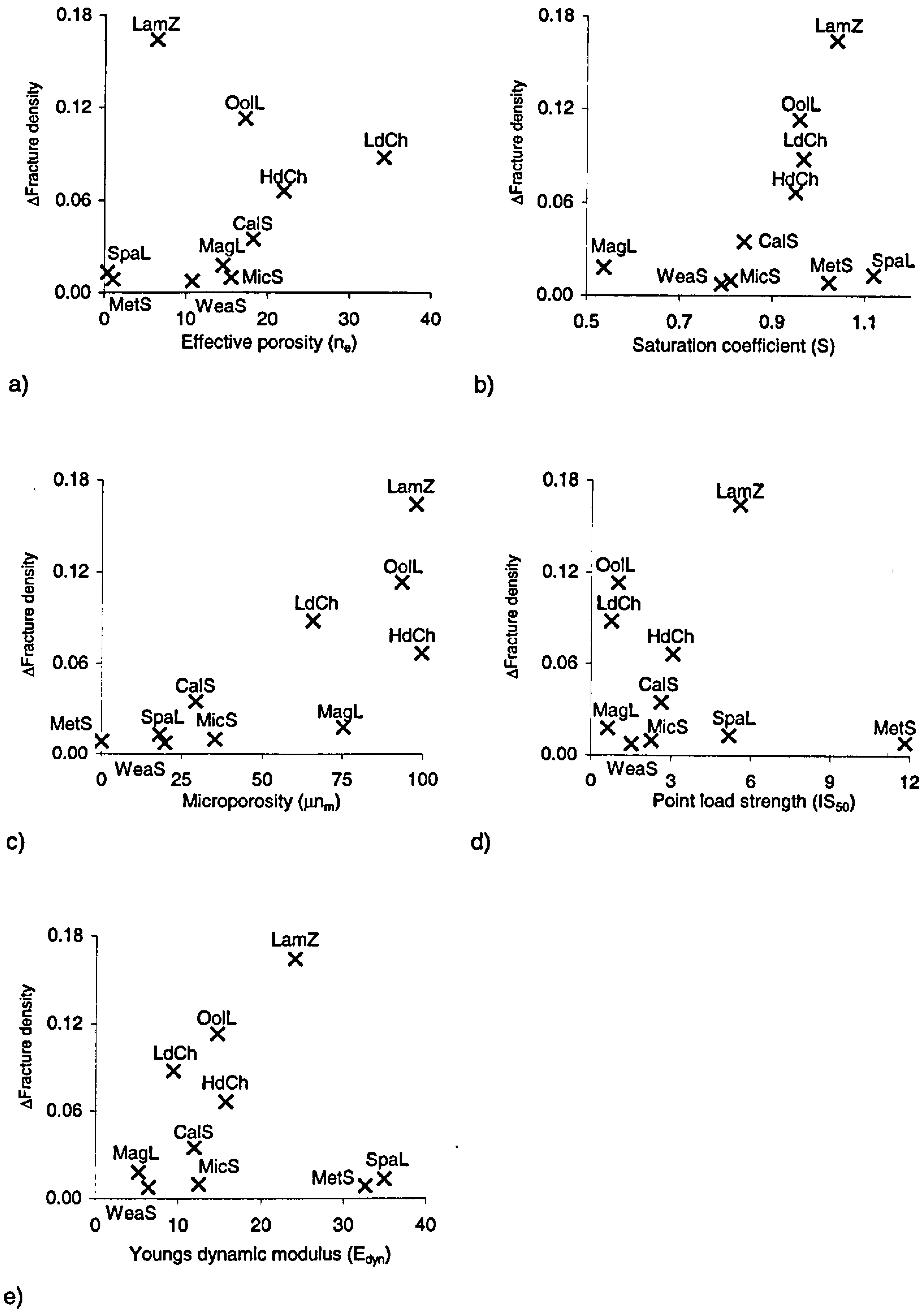


Figure 4.20 (a-e) Correlation of rock properties with Δfracture density for the freeze-thaw test

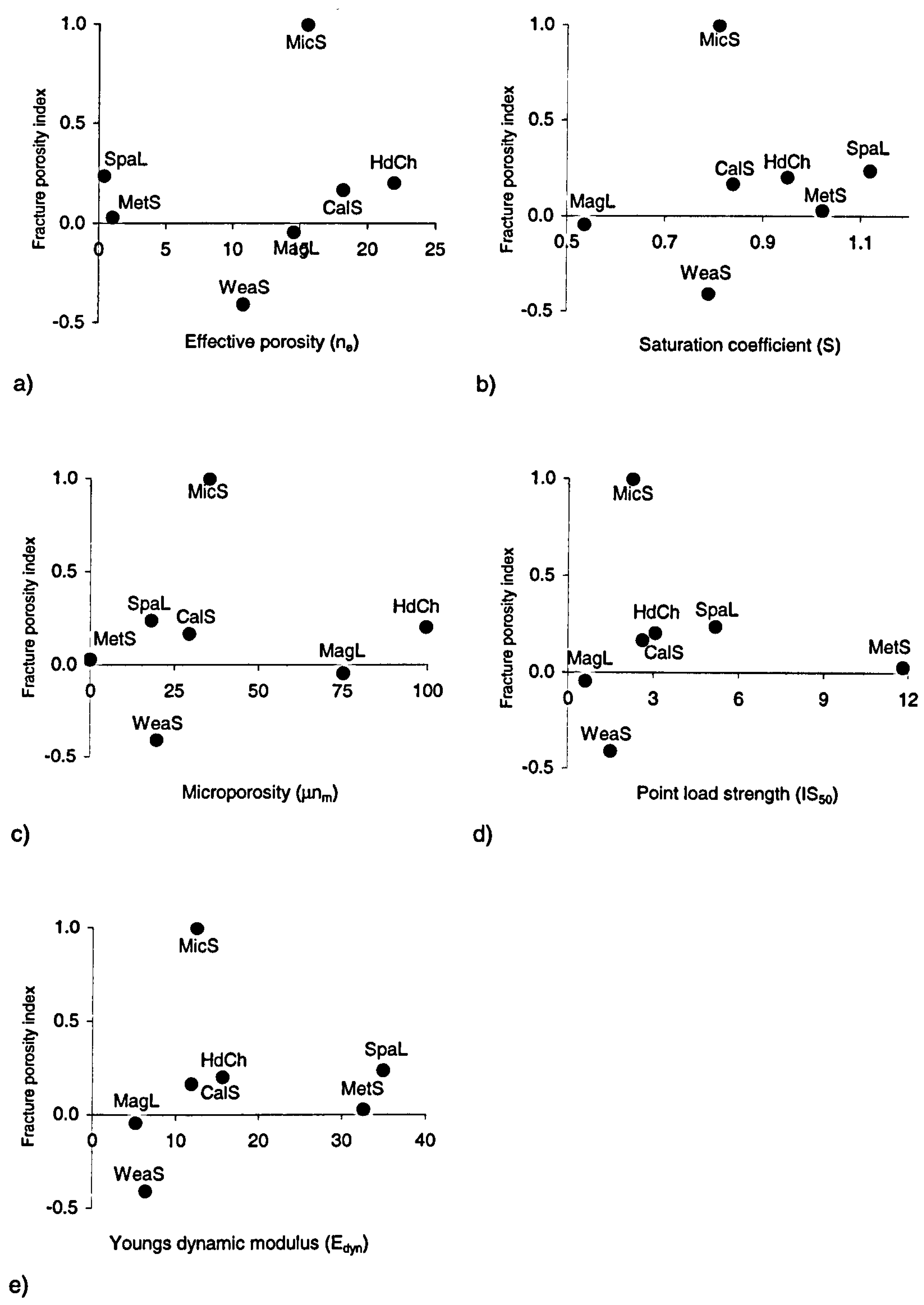


Figure 4.21 (a-e) Correlation of rock properties with fracture porosity index for the freeze-thaw test

properties (Figure 4.20d and 4.20e) reveals a similar distribution of points except that for LamZ a much greater degree of fracturing is seen compared to other rocks with a similar strength (SpaL). There is no clear trend between fracture density and elasticity.

The charts showing fracture porosity exclude LamZ and OolL(2) because of the high mean values involved (11.84 and 2.35 respectively) and it was not possible to obtain a value for LdCh (Figure 21 a to e). There does not appear to be any correlation between fracture porosity and rock properties. For MagL, HdCh, WeaS and CalS only, there is a trend for increasing n_e with increasing IF_p, but it is not possible to identify any common characteristic between these rocks which might explain this.

Post-freeze-thaw test fracture porosity plotted against compressive strength for all individual specimens is shown in Figure 4.25. While there is no linear correlation between parameters, three distinct groups of data points can be identified. (i) The strongest rocks (red line) show no significant change in fracture porosity. This is expected since these rocks have the greatest resistance to deterioration. (ii) The weakest rocks (blue line) show predominantly negative values and low positive values. A negative fracture porosity indicates closure of pores and microcracks might have occurred in the weaker rocks due to application of the weight (see section 3.4.3.5 in Chapter Three) or other deformation of void spaces due to ice crystallisation. (iii) The remaining, moderate strength rocks (green line) show small increases in fracture porosity indicating the disruptive effects of freeze-thaw.

4.6.2 Salt weathering

Weight loss results for the salt weathering test reflect three categories of response relating to infilling of pores with salt deposits. These are: (i) rocks which resisted any change (SpaL, MetS); (ii) rocks in which weight *gains* occurred (OolL, HdCh, CalS); (iii) rocks in which pore infilling either did not occur or which was disguised by larger reductions in weight due to breakdown (LdCh, MagL, WeaS, MicS, LamZ). As such, meaningful correlation between weight loss and pore-dependent properties cannot be discerned.

There is no clear correlation between effective porosity and weight loss (Figure 22a) but a positive correlation with fracture density (Figure 23a). As for freeze-thaw, LamZ is anomalous because of the extremely high density of fracturing which occurred. For some rocks, notably MagL, MicS, WeaS, LdCh and LamZ, there appears to be a positive correlation between weight loss and S (Figure 22b), but for the remaining rocks, weight loss is similar despite considerable variation in S. If MagL, SpaL and MetS are removed from the chart (see earlier discussion) a clearer positive correlation with fracture density becomes apparent (Figure 23b).

A positive correlation exists between microporosity (μn_m) and weight loss for a number of rocks (Figure 22c), though MagL, OolL and HdCh have very low weight loss despite similar μn_m to LdCh and LamZ. Again, the correlation is a little stronger for fracture density (Figure 23c).

The distribution of data points for IS_{50} and E_{dyn} is similar, reflecting their interdependence, but a clear relationship with weight loss and fracture density is not discernible (Figure 22d, 22e, 23d

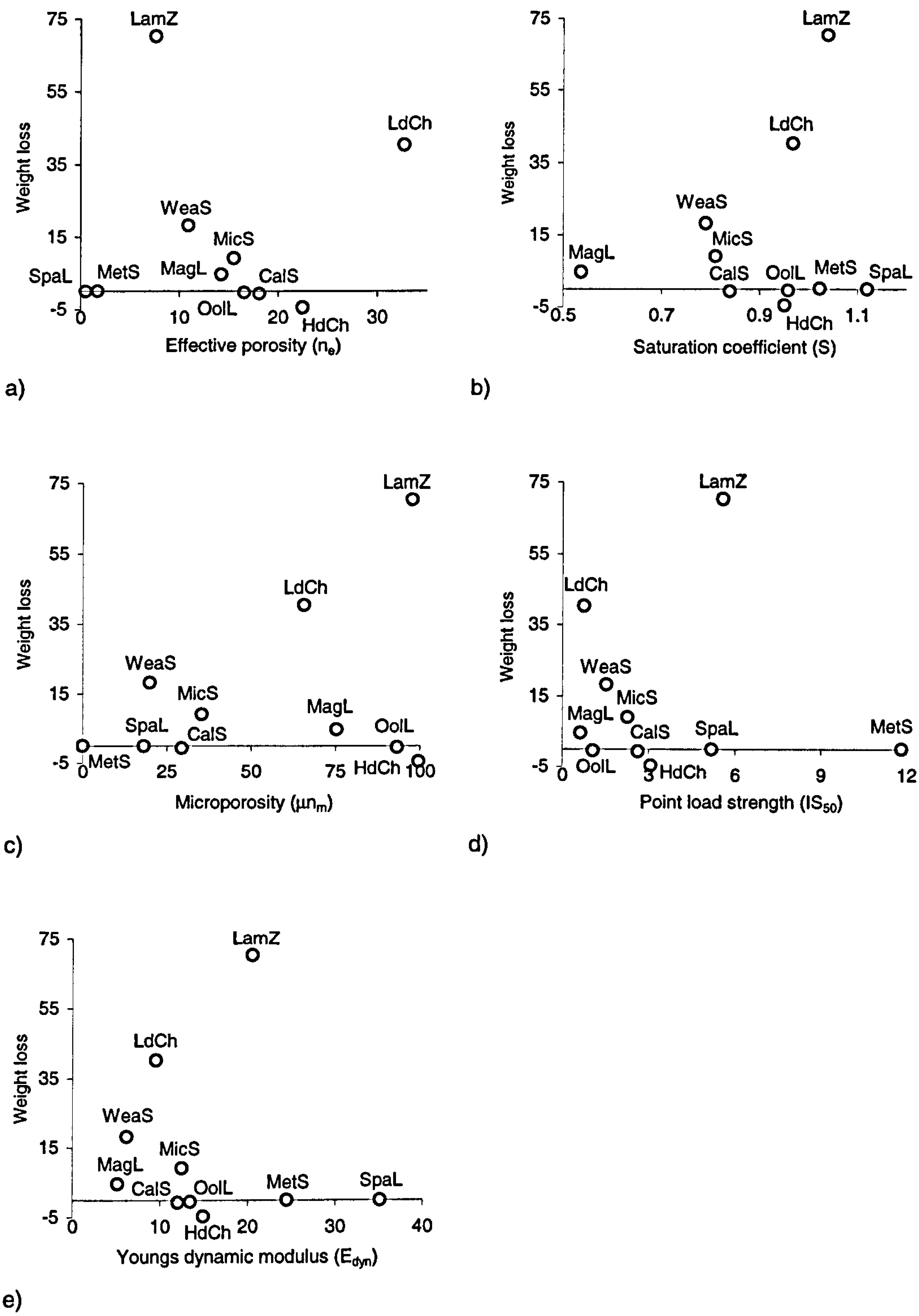


Figure 4.22 (a-e) Correlation of rock properties with weight loss for the salt weathering test

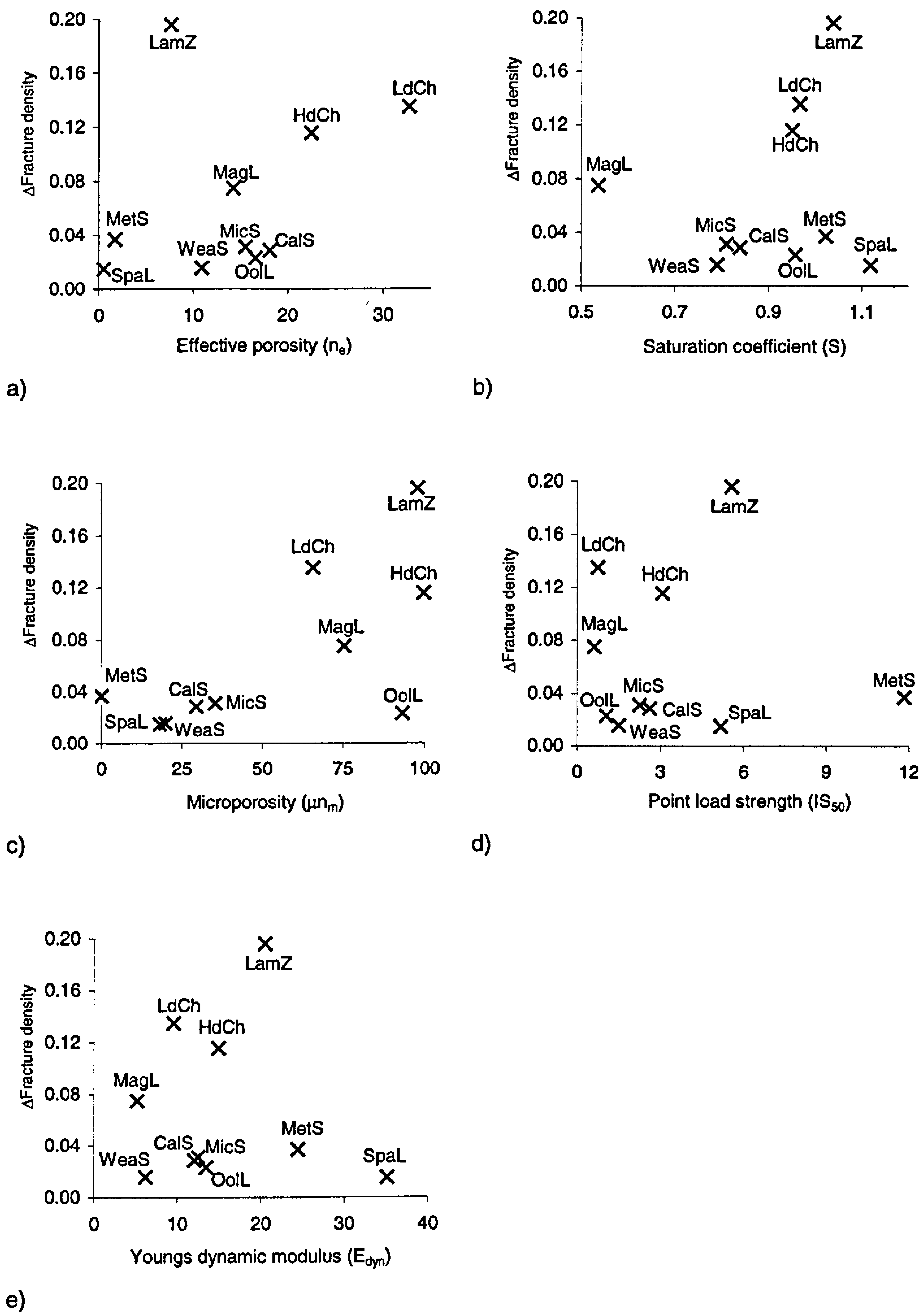


Figure 4.23 (a-e) Correlation of rock properties with Δfracture density for the salt weathering test

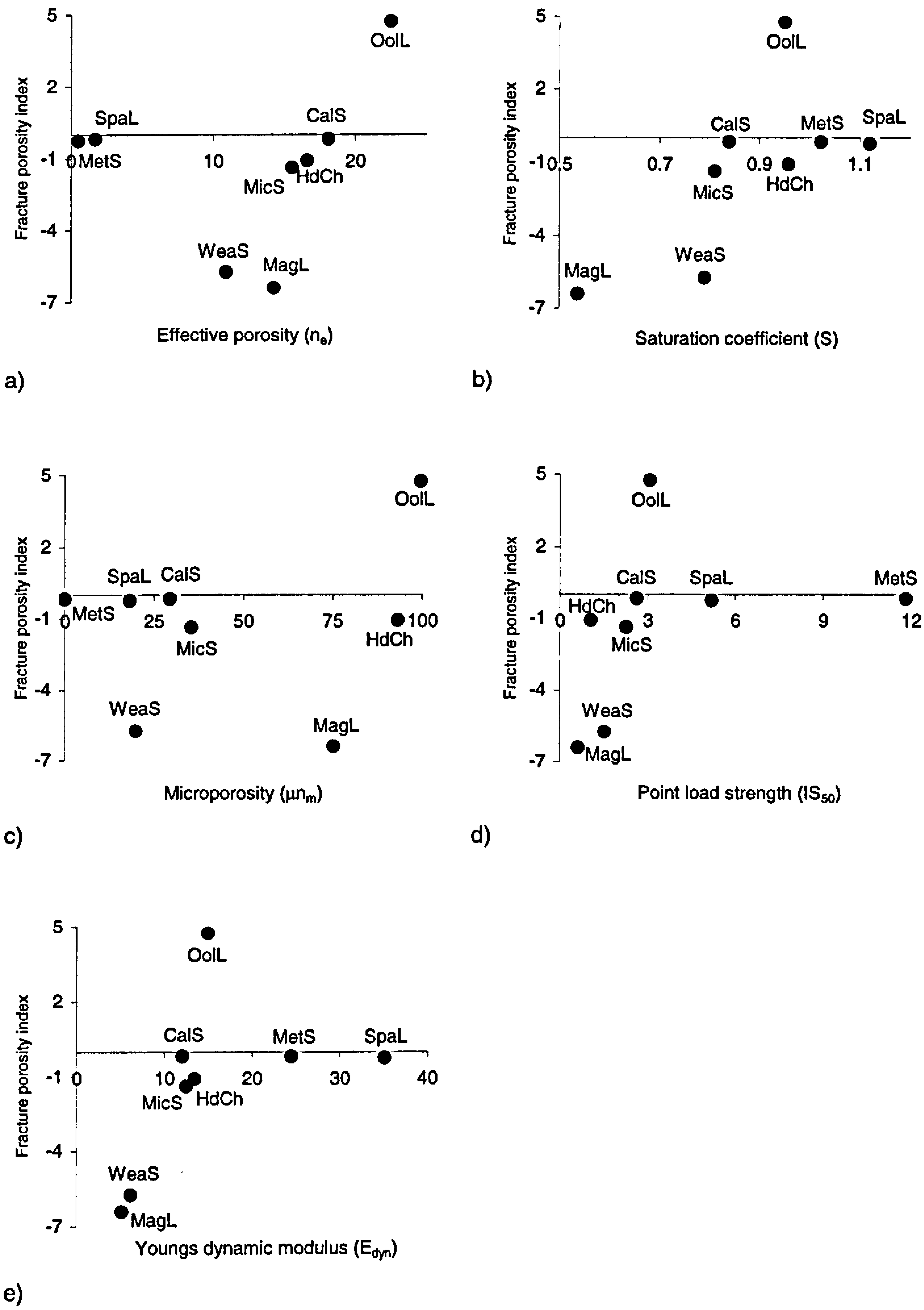


Figure 4.24 (a-e) Correlation of rock properties with fracture porosity index for the salt weathering test

and 23e). An increase in deterioration with weaker, less elastic rocks would be expected but only a hint of this is seen in the charts with a number of points, notably LamZ, being anomalous.

A positive correlation is evident between fracture porosity and n_e, S, IS_{50} and E_{dyn} (Figure 24a, b, d and e) though SpaL and MetS are anomalous in all of these cases. There is no discernible trend for μn_m (Figure 24c). Figure 4.27 shows post-salt weathering fracture porosity plotted against compressive strength for all individual specimens. The same three distinct groups of data points identified for freeze-thaw are also evident here (refer to 4.6.1 above)

4.6.3 Wetting and drying

It is not appropriate to attempt any statistical analysis for the wetting and drying tests where only four rocks were tested, particularly since the deterioration which occurred for three out of the four rocks was negligible.

4.6.4 Slake durability

Broadly, slake durability tended to increase with lower n_e and μn_m (Figure 4.27a and c) and increasing IS_{50} and E_{dyn} (Figure 4.27d and e). However, a number of anomalies can be identified. For instance, OolL, MagL, CalS and HdCh have higher n_e, yet are a little more resistant to slaking than are LamZ, WeaS and MicS. In contrast, MagL and WeaS have a similar E_{dyn} to LdCh but a much higher slake durability. A further anomaly relates to the contrast in slake durability of LamZ and SpaL, despite very similar values for IS_{50}. There is no clear relationship between slake durability and S (Figure 27b).

4.6.5 Discussion

A greater number of samples would be needed before definite conclusions could be drawn about the relationships between deterioration and the rock properties examined here. Nevertheless, some trends are discernible and the implication of these is discussed further.

For all of the weathering tests conducted there is a general trend for increasing susceptibility with increasingly connected pore volume. Rocks with particularly low porosity (eg SpaL and MetS) consistently experienced least deterioration, evidence that the disruptive effect of ice or salt crystal growth is minimised due to an absence of pore fluid (McGreevy 1996). Conversely, rocks with high porosity (eg LdCh) suffered significant damage, evidence that water absorption maximises the capacity for disruption due to weathering. For these rocks, it might be possible to estimate likely response to weathering on the basis of effective porosity alone. However, for other rocks and for significant anomalies, the influence of other rock properties is indicated. It is also recognised that total pore volume correlates reasonably well with increasing mechanical strength.

The trend for increasing durability with decreasing S, as found here, was explained by Everett (1961) as a function of the fact that with low S, or with S below some critical threshold level, ice or salt crystallisation pressure could be accommodated by extrusion into unfilled pores. This

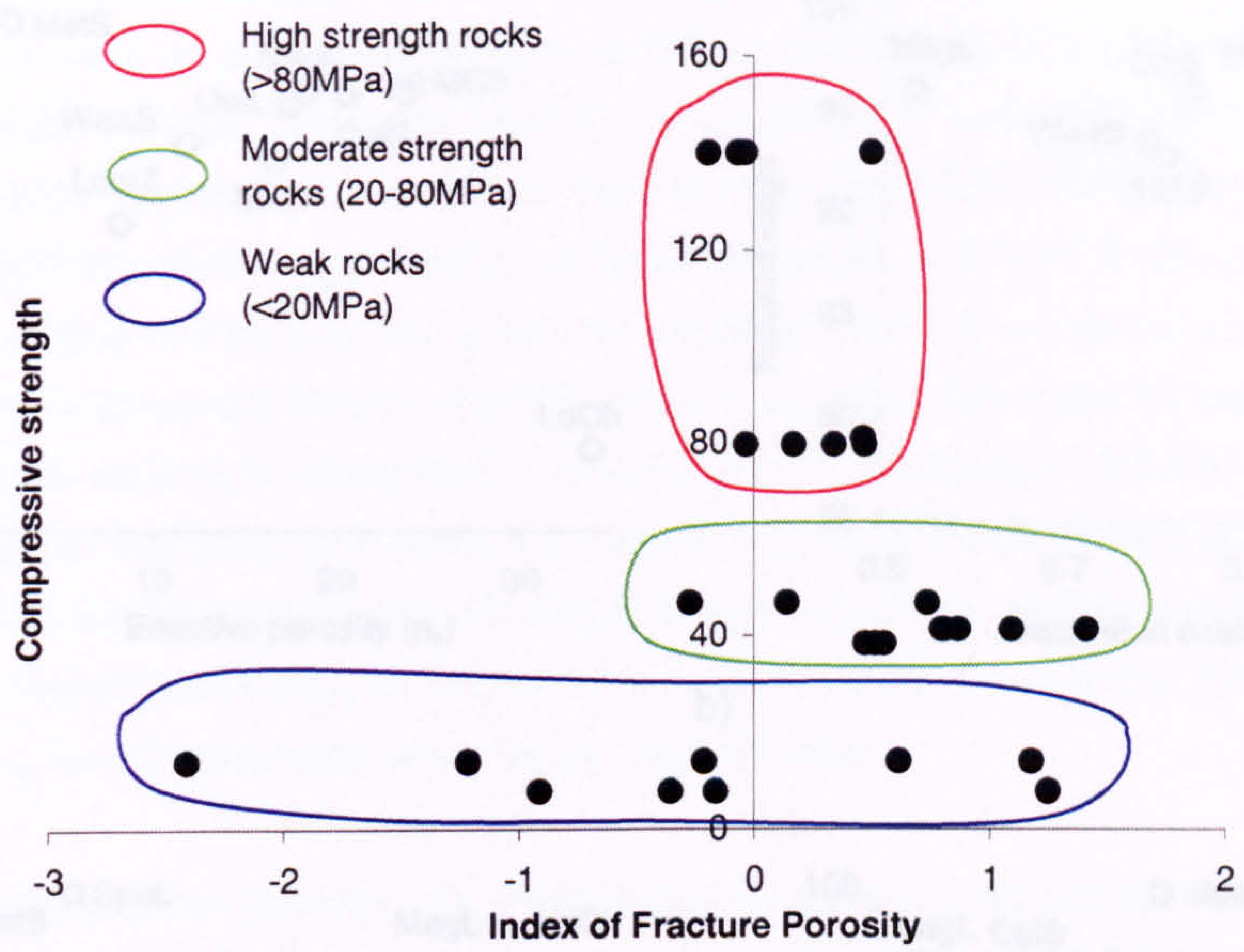


Figure 4.25 Relationship between compressive strength and fracture porosity for freeze-thaw

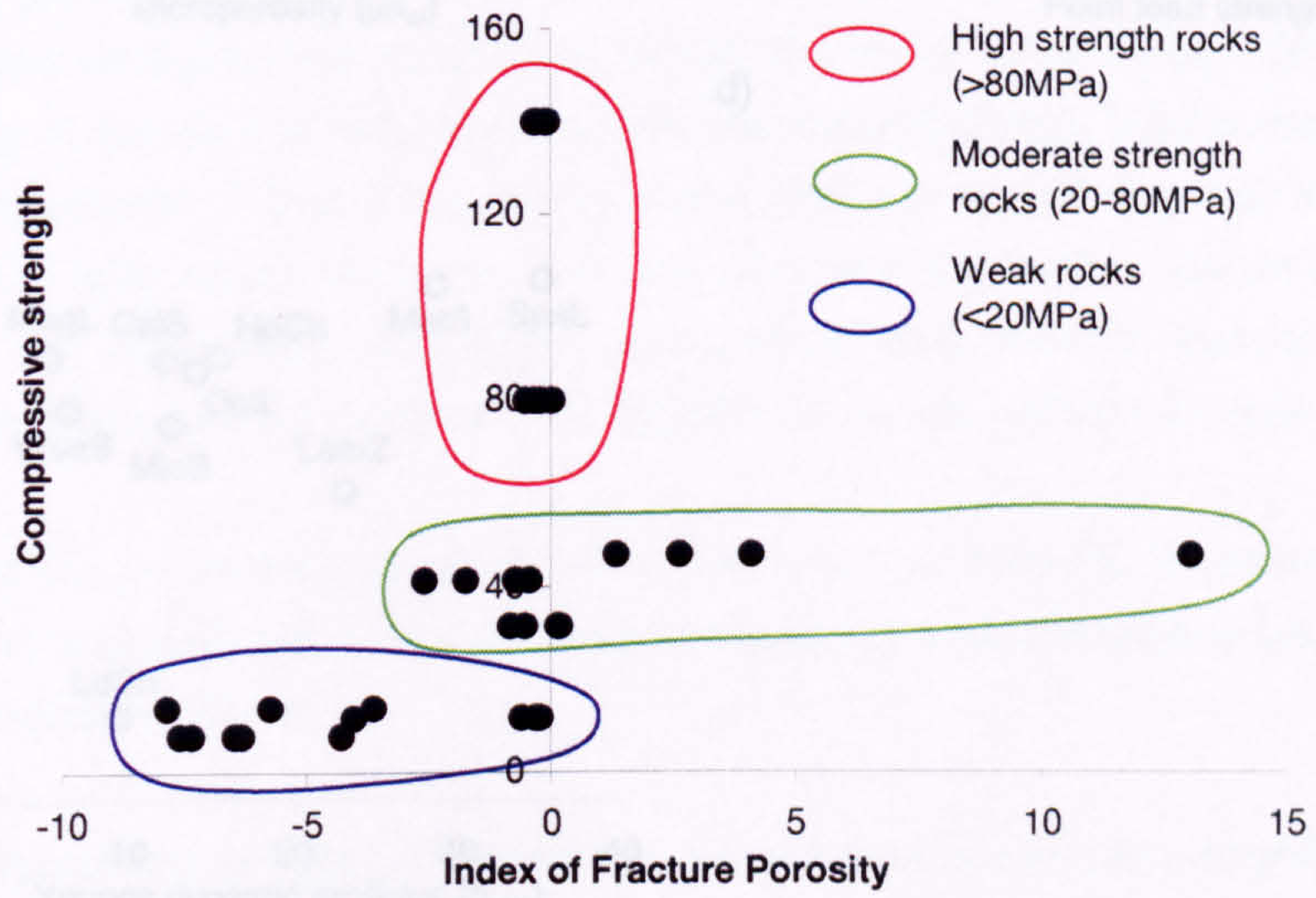


Figure 4.26 Relationship between compressive strength and fracture porosity for salt weathering

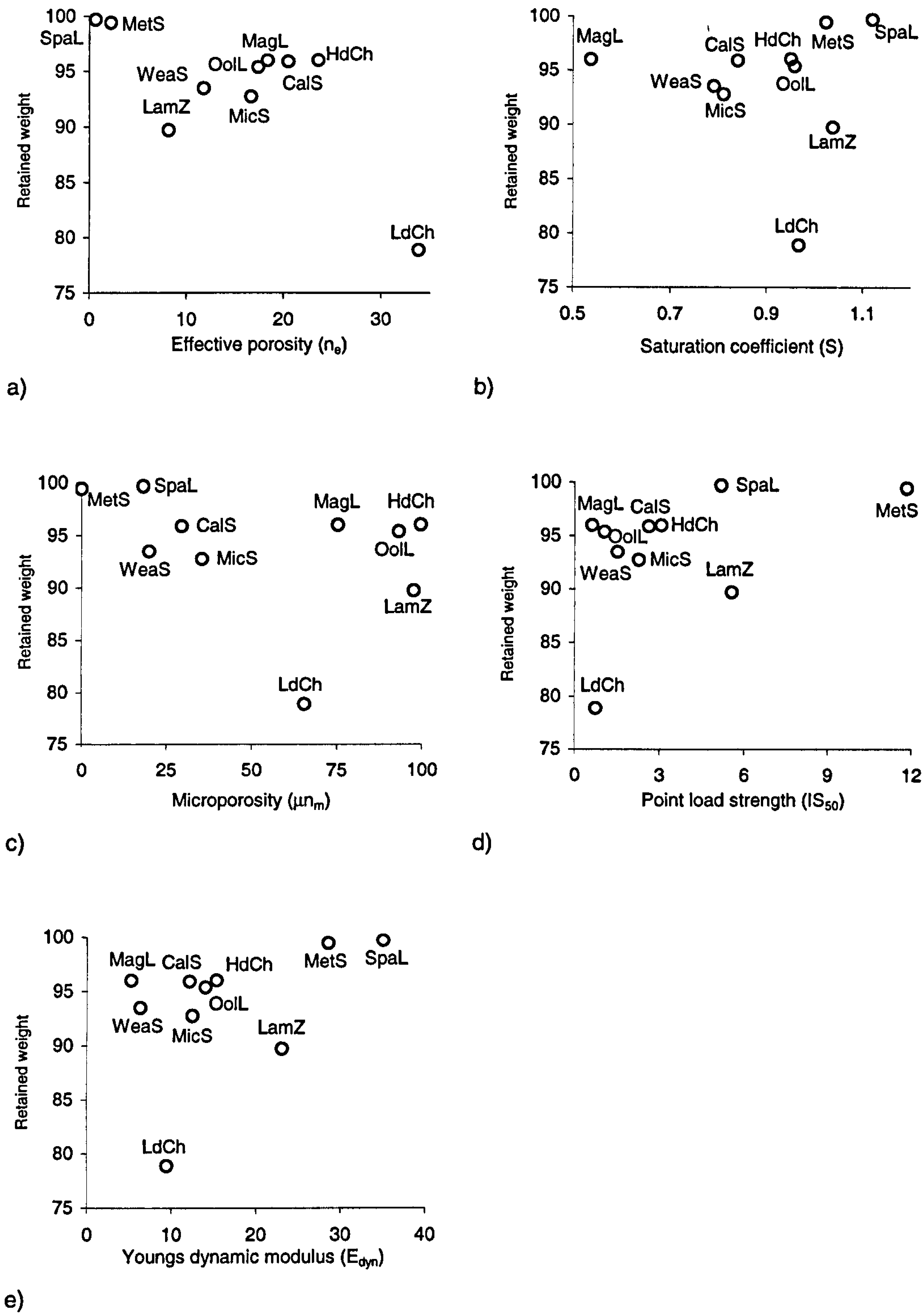


Figure 4.27 (a-e) Correlation of rock properties with % retained weight for the slake durability test

trend was also observed by Hall (1988a) and Richardson (1991). The latter found that rocks with a higher proportion of coarse pores have higher durability. He related durability to moisture absorption into the rock, and "room to accommodate crystal growth" (p 23). For the freeze-thaw and salt weathering tests there appears to be a threshold level for saturation coefficient of around 0.85, above which there is a rapid increase in susceptibility. This figure fits well with the saturation threshold values suggested by Kreuger (1923) of 0.85 and Hirschwald (1912) of 0.80. However, these figures relate to the volumetric expansion of ice on freezing and so it is difficult to see why similar threshold levels should be identified in the data here for salt weathering tests as well as freeze-thaw. It is more likely, as suggested by McGreevy (1996), that rather than S exerting any direct influence of its own, its importance lies in the fact that it is a broad reflection of pore structure and microporosity. Although McGreevy (1982, 1996) found a similar positive correlation between S and μn_m, as found here (see discussion in section 4.4.3), he found that decreasing μn_m and S correlated with decreasing durability.

Similarly, the index of fracture porosity reflects real change in void space due to weathering, as well as blockage of pores due to salt deposition. A positive correlation between fracture porosity and S is identifiable. This indicates that greater generation of new void is associated with increasing degree of saturation, perhaps because there is very little unfilled pore space to be taken up by crystallisation pressures. Salt deposition in pores either does not occur or its effects are masked by the sample damage. Conversely, with a low percentage saturation, the negative values for IF_p suggest deposition in pores is more likely.

It has been argued (Price 1978) that rocks with a high μn_m might not undergo such severe test conditions because being fine-pored, they would dry much more slowly. Observation of the surface drying of samples tested here supports the contention that finer-pores rocks dry more slowly. The suggestion is that if the drying phase of a salt weathering test is incomplete, full crystallisation of salts would not occur. Thus the pore size distribution has an indirect influence on durability. For other tests, incomplete drying would also result in incomplete moisture or thermal cycles being applied. However, this hypothesis can be rejected in three grounds:

(i) In the current testing, samples were oven-dried to 105±2°C to constant weight. It is difficult to accept that at this extreme of drying salt crystallisation would not occur, even in the smallest pores.

(ii) The hypothesis suggests that high μn_m should equate with high durability, when for the rocks tested here, the reverse is true.

(iii) Experimental work by McGreevy (1982) found no clear relationship between drying rate and susceptibility to salt weathering.

Compressive strength and Young's modulus are closely related (Deere and Miller 1966) and this can be seen by the similarity for all tests of correlation between durability and IS_{50} and E_{dyn}, the former also being closely related to compressive strength (Broch and Franklin 1972). The relationship between IS_{50} and E_{dyn} and deterioration is not clear-cut, however, and this is probably because they both reflect a wide range of rock properties including mineralogy and cementing material, texture, density and total pore volume (Allison and Goudie 1994).

It is clear that no single property can be used to explain susceptibility of these rocks to the weathering tests conducted. It is also clear that for some rock properties the relationship to weathering susceptibility varies with the deterioration indicator. Findings suggest that n_e and μn_m are of most significance in controlling breakdown but that void dependent properties are difficult to interpret for the salt weathering test because of pore infilling. The relationship between μn_m, S and deterioration also indicates that general pore size is inextricably tied to the weathering mechanisms operating. Further insight into this relationship could be obtained from detailed analysis of the pore structure distribution rather than the somewhat arbitrary cut-off imposed in determination of μn_m.

CHAPTER FIVE
MECHANISMS OF ROCK BREAKDOWN DUE TO EXPERIMENTAL WEATHERING

5.1 Introduction

In this chapter the results of the experimental weathering programme presented in Chapter Four are explored further. First, the emphasis is on describing the mode of deterioration at the hand specimen scale and evaluating the role of macro flaws in breakdown. The chapter begins with a classification of rock flaws based on pre-test observations of the samples used. In the second part of the chapter, the emphasis changes to an evaluation of the nature of breakdown mechanisms, primarily at the micro scale. This is achieved by analysis of modifications to rock properties brought about by experimental weathering.

5.2 Classification of Rock Flaws

On the basis of the rocks investigated here, a simple classification of pre-existing rock flaws has been developed and is given in Figure 5.1. This is based on careful observation and description of the features seen in hand specimen. Table 5.1 indicates those rock flaws which were present in each sample prior to testing and flaws indicated in bold and underlined are those which appeared to be most closely associated with deterioration. A brief description of the mode of deterioration is also given though this is covered in greater detail in section 5.3. Reference is made in the descriptions to the relationship between rock flaws and deterioration mode which is also covered in greater depth in section 5.4.

5.3 The Mode of Deterioration

In this section, detailed descriptions of the mode of breakdown are given for each sample with respect to each weathering test. For each rock type, a pictorial record is provided, given in Figures 5.2 to 5.11. These illustrate the visible deterioration which occurred in hand specimens for representative specimens of each rock type. Special attention is given in annotations to the relationship between deterioration and pre-existing rock flaws.

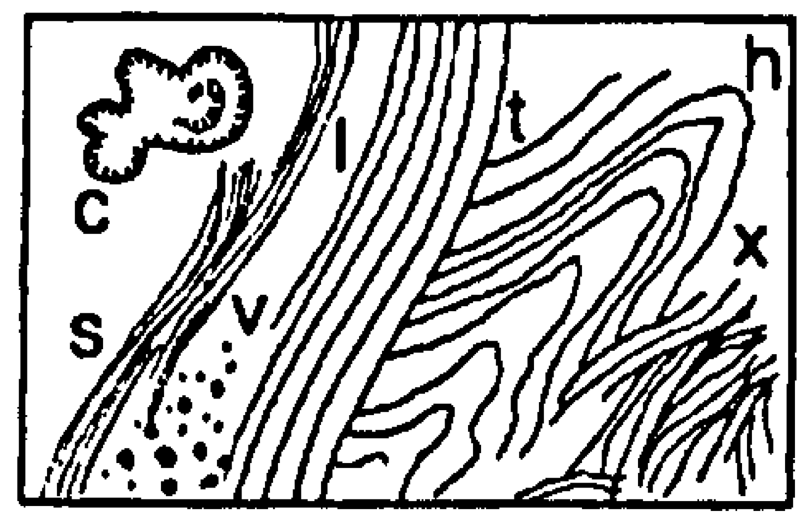
5.3.1 Freeze-thaw

Low density chalk (LdCh) (Figure 5.2a): All specimens deteriorated rapidly and severely, disaggregating into many small fragments and creating a large amount of debris and fines (Plate 5.1). Much of the initial loss of fragments related to the rough textured, higher porosity zones identified prior to testing (section 3.7.1 of Chapter Three). This is not surprising given the close association of frost susceptibility with effective porosity. Even after just six cycles, no original rock surface remained and many new fractures had developed, commonly curved, sub-parallel and with wide apertures. Some curved fractures relate to shear structures in the rock. Shallow scaling with sharp, angular boundaries was also common and was sometimes

PRIMARY FLAWS

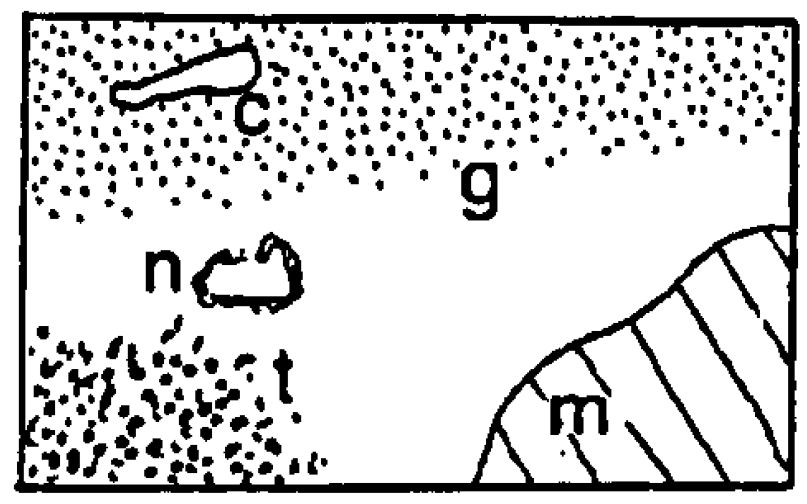
Primary Depositional Structures (S)

- laminations (**l**), truncated surface (**t**), cross laminae (**x**), and fold hinges (**h**);
- shear and deformation structures (**s**);
- syn-depositional cavities (**c**) and voided zones (**v**).



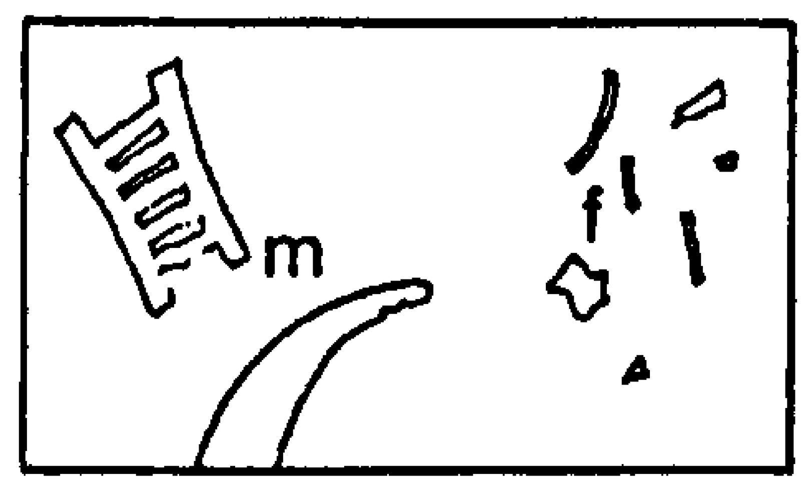
Lithological Variations (L)

- grain size variations or boundaries (**g**);
- other variations in porosity or surface texture (**t**);
- variations in mineralogical composition (**m**);
- lithic clasts (**c**) or nodules (**n**) (may be secondary)



Other Primary Flaws (O)

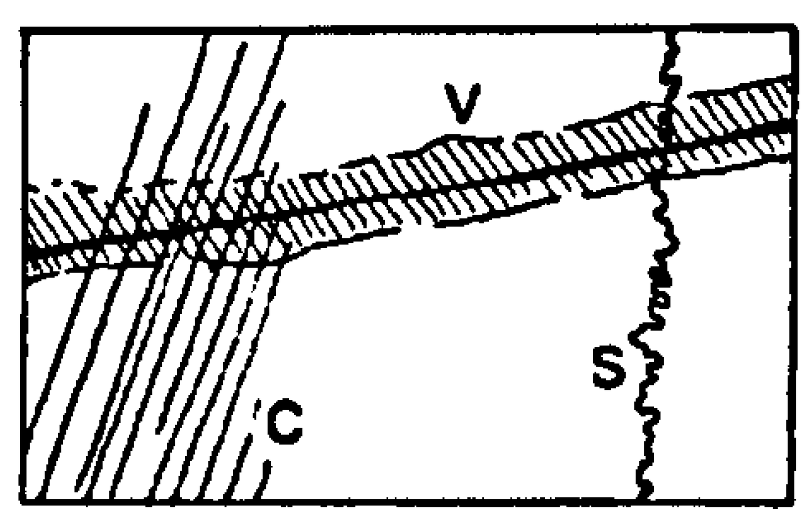
- macro fossils (**m**) and shell fragments (**f**)



SECONDARY FLAWS

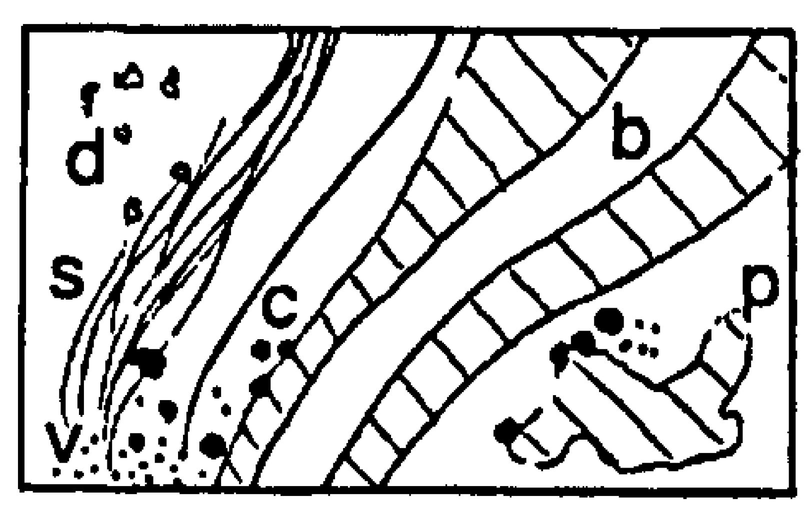
Diagenetic and Metamorphic Effects (D)

- mineral veins and healed fractures (**v**);
- stylolites and other pressure solution features (**s**);
- cleavage (**m**), foliation (**f**) or banding (**b**).



Weathering Effects (W)

- discoloured: spots (**d**), patches (**p**) banding (**b**) or streaking (**s**);
- solutional or physical removal of material leaving cavities (**c**) or voided zones (**v**) - also in **S** above.



Fractures (F)

- part open fractures (**o**) eg with rock bridges;
- weak incipient fractures (**w**);
- strong incipient fractures (**s**).

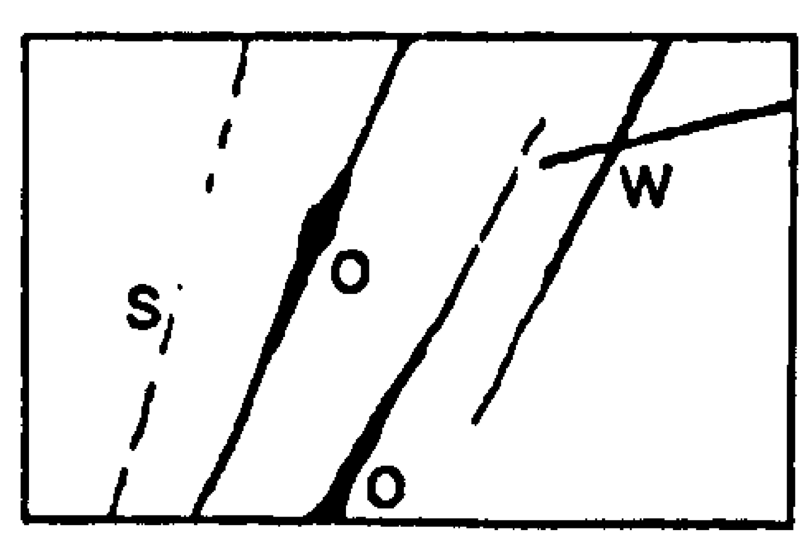


Figure 5.1 Classification of pre-existing flaws

associated with non-penetrative flaws such as macro fossils and weathered or discoloured zones. By the end of testing, wholesale disaggregation had occurred, though larger individual fragments contained scales and open cracks within them. The relationship between deterioration and pre-existing flaws was particularly difficult to determine for this sample due to the rapidity and severity of breakdown.

SAMPLE	PRE-TEST FRACTURE DENSITY*1	PRE-EXISTING FLAWS*2	SUMMARY OF DETERIORATION MODE FOR EACH WEATHERING TEST*3
Low density chalk (LdCh)	3.9 *(5.7)*	**Ss**, Sv, **Lt**, **Om**, Of, Dv, **Ds**, Wd, Wp, **Ws**, Wv, Fw	**FT**: Rapid and severe disintegration; shallow scaling; intense fracturing; **MS**: severe disintegration; intense fracturing; multiple flaking; surface pitting; **WD**: negligible deterioration; **SD**: severe granular loss
Magnesian limestone (MagL)	5.7 *(7.8)*	**Ds**, Wd, **Wb**, Wc, Wv, **Fo**, **Fw**, **Fs**	**FT**: Fracturing; minor fragmentation, scaling and cavity development; **MS**: scaling; fracturing; cavity development; minor fragmentation; **SD**: cavity development; fracturing; minor granular loss and fragmentation
Oolitic limestone (OolL)	2.0 *(3.6)*	**Sc**, **Sv**, **Om**, **Of**, Wd, **Fs**	**FT**: Rapid and severe disintegration; intense fracturing; deep scaling; **MS**: cavity development; fracturing; minor granular loss and fragmentation; **SD**: granular loss
High density chalk (HdCh)	2.3 *(4.2)*	**Om**, Of, Dv, **Ds**, Wp, Wc, **Fs**	**FT**: Fracturing; deep scaling; breakage; **MS**: intense incipient fracturing; surface pitting; minor breakage; **WD**: no deterioration; **SD**: negligible deterioration
Sparry limestone (SpaL)	13.9 *(7.2)*	Ss, Lm, **Om**, Of, **Dv**, Ds, Wd, Ws, **Fs**	**FT**: Minor fracturing and fragmentation; rare breakage; **MS**: Minor fracturing and fragmentation; **SD**: no deterioration
Weathered sandstone (WeaS)	0.0 *(0.0)*	**Lc**, Wd, Wb, Ws	**FT**: Granular loss; minor fracturing; **MS**: granular loss; minor fragmentation, breakage and fracturing; **SD**: granular loss; minor fracturing
Calcareous sandstone (CalS)	0.9 *(1.4)*	Sl, **Lt**, **Lm**, Of, Ws	**FT**: Fracturing; scaling; minor granular loss; **MS**: surface pitting; minor scaling and fracturing; **WD**: no deterioration; **SD**: minor granular loss
Micaceous sandstone (MicS)	0.7 *(1.4)*	Sl, St, **Sh**, **Lg**, **Lm**, **Lc**, Ln, **Fs**	**FT**: Granular loss; minor fracturing; **MS**: surface pitting; granular loss; scaling; fracturing; breakage; **SD**: granular loss
Laminated siltstone (LamZ)	22.2 *(13.0)*	**Sl**, St, Sx, Sh, Ss, Lg, **Fo**, **Fw**, **Fs**	**FT** and **WD**: Severe fracturing; multiple flaking; breakage; **MS**: severe disintegration and fracturing; **SD**: granular loss; fracturing; minor fragmentation and breakage
Metasediment (MetS)	28.2 *(14.3)*	**Sl**, **Dm**, Ws, **Fs**	**FT** and **MS**: Minor fracturing; **SD**: Negligible deterioration

Table 5.1 Rock flaws observed in the test samples and their role in deterioration

Note 1 Fracture density is given as $x10^{-3}$ mm^2/mm^3 with standard deviations in parenthesis

Note 2 Flaws in bold and underlined were most closely associated with deterioration

Note 3 Where FT = freeze-thaw; MS = salt weathering; WD = wetting and drying; and SD = slake durability

Magnesian limestone (MagL) (Figure 5.3a): Five deterioration modes for MagL were identified: (i) Many new, angular, sub-parallel, axial cracks formed, commonly parallel to pre-existing cracks. (ii) Pre-existing cracks also became extended. (iii) Minor fragmentation occurred, especially in association with pre-existing cracks, and occasionally also with stylolites. (iv)

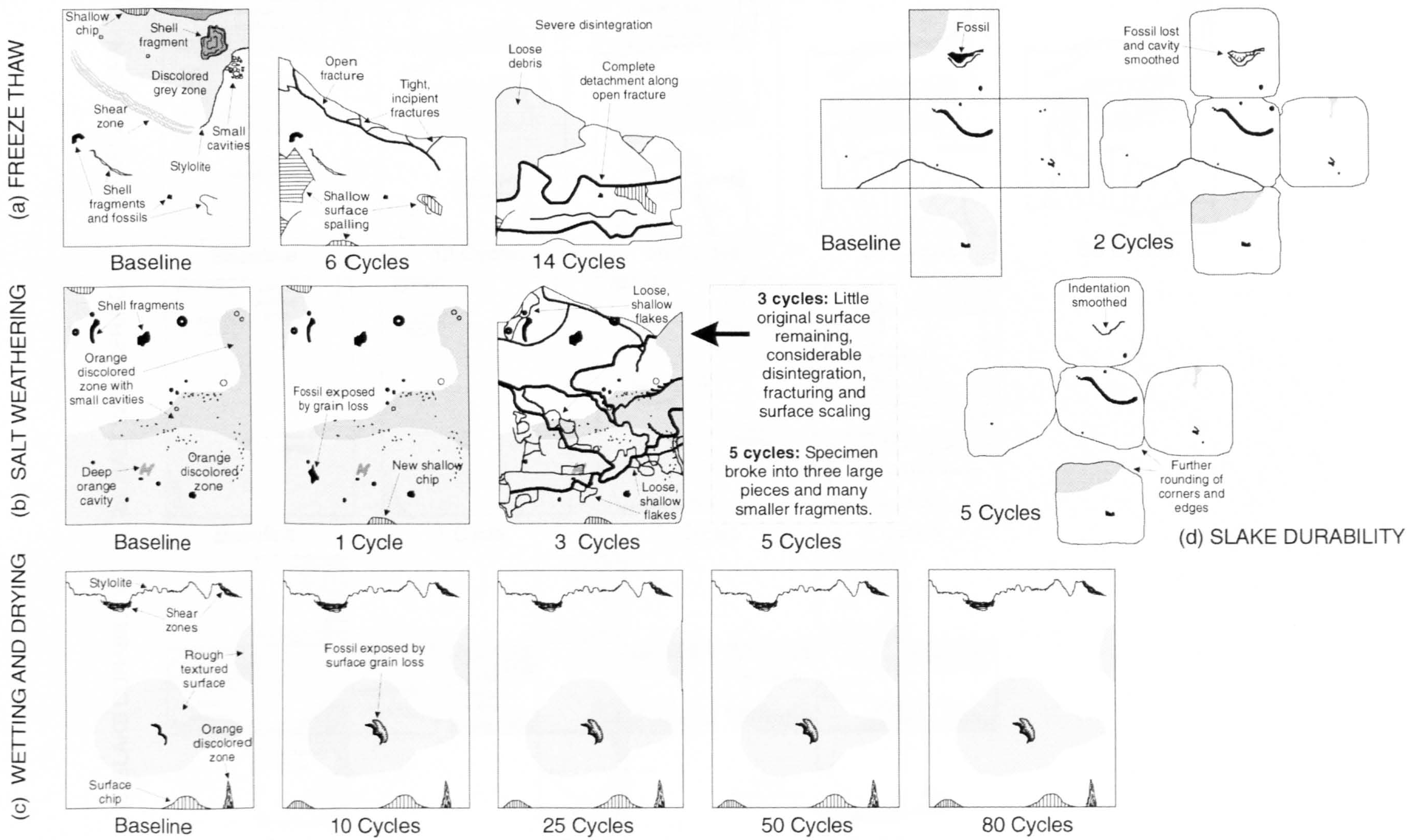


Figure 5.2 Pictorial record of deterioration for the low density chalk (LdCh)

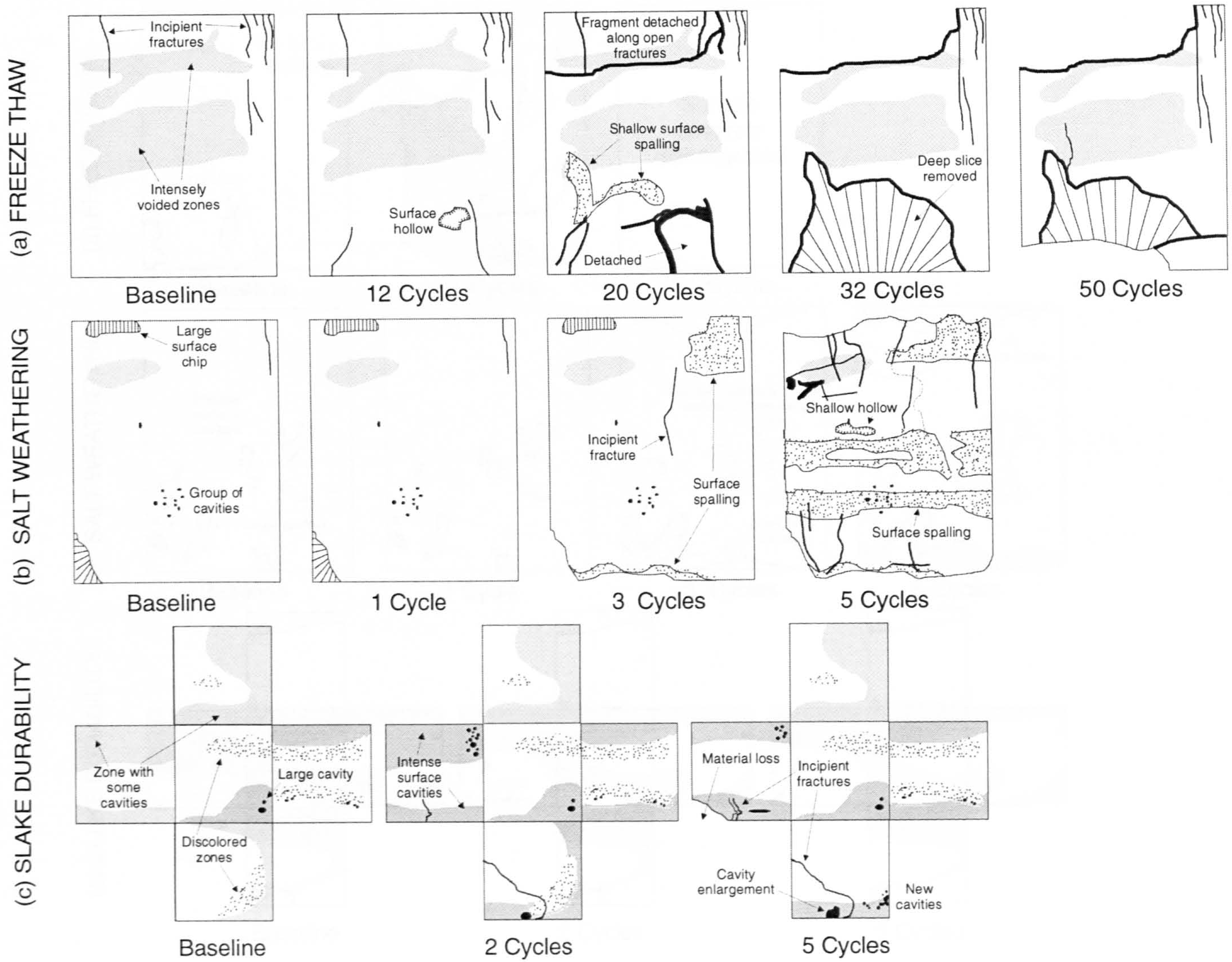


Figure 5.3 Pictorial record of deterioration for the magnesian limestone (MagL)

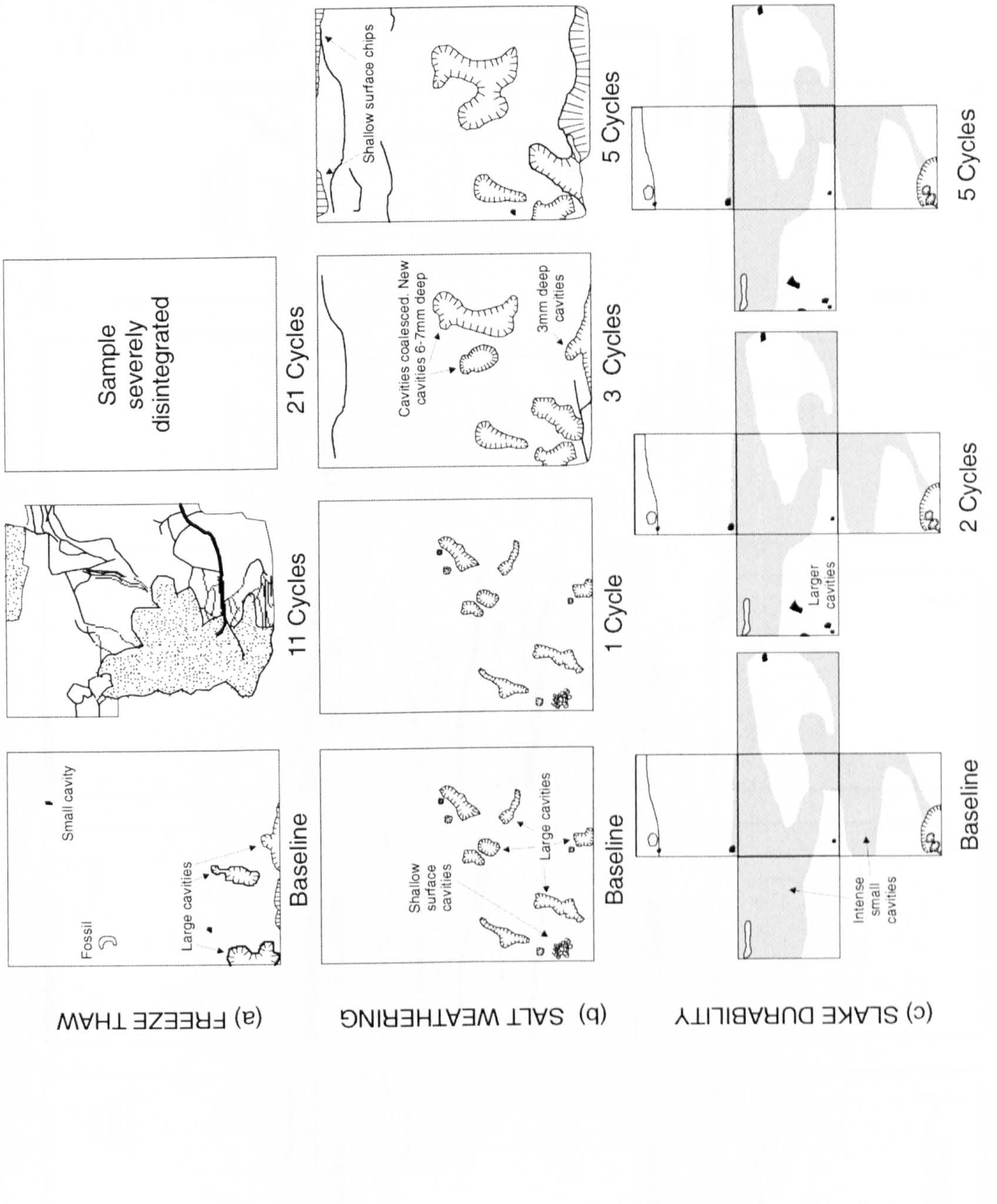


Figure 5.4 Pictorial record of deterioration for the oolitic limestone (OolL)

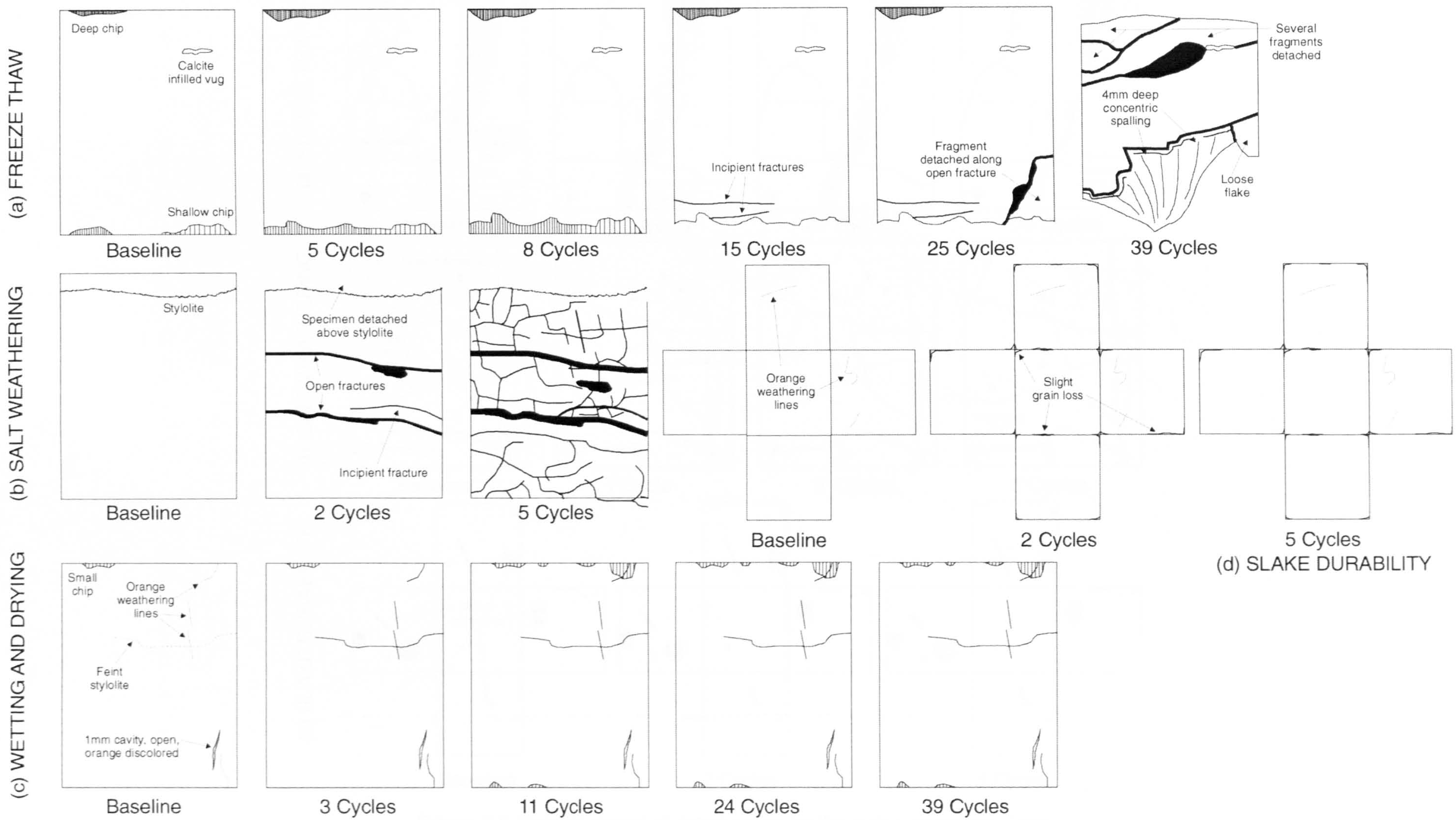


Figure 5.5 Pictorial record of deterioration for the high density chalk (HdCh)

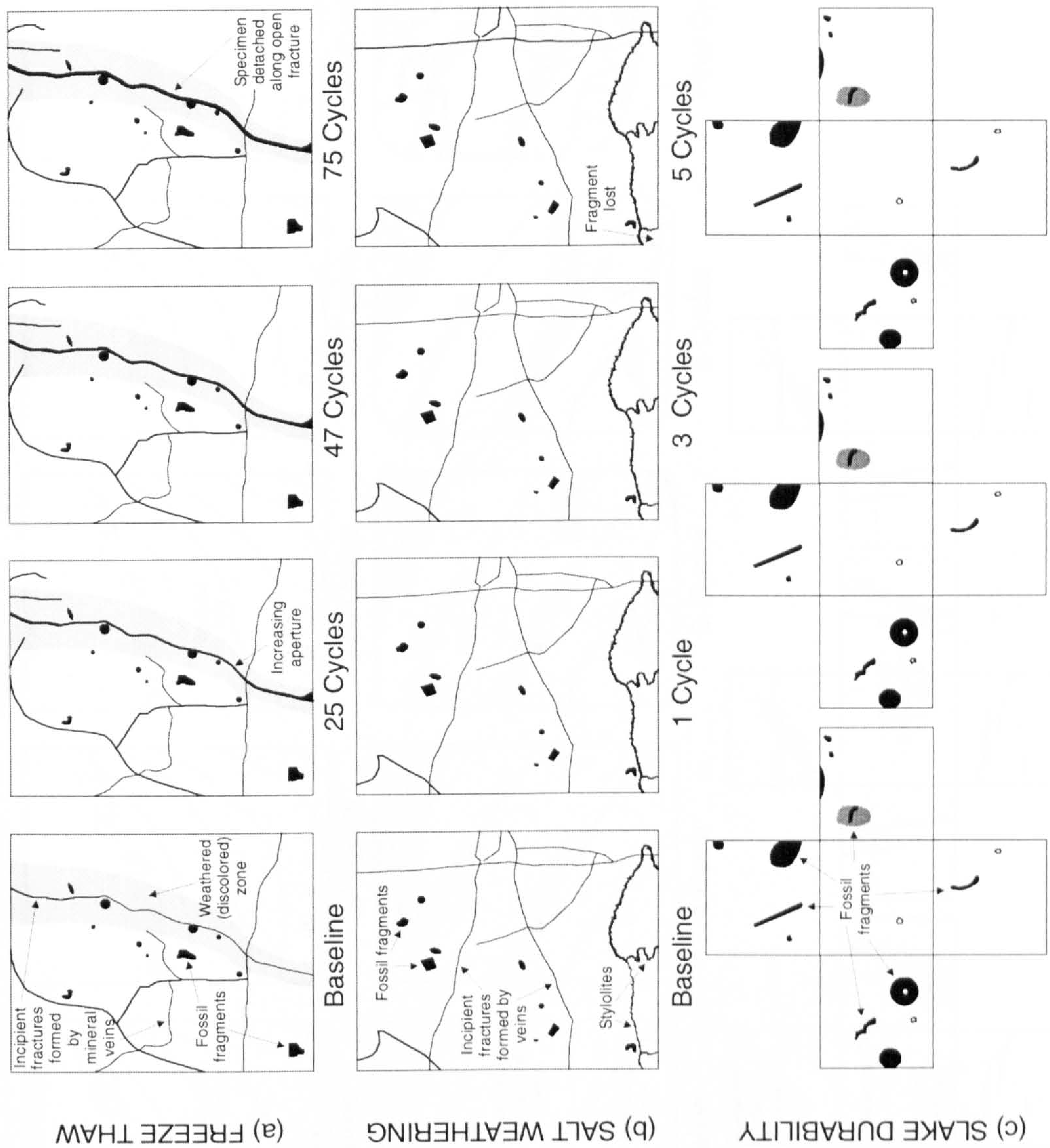


Figure 5.6 Pictorial record of deterioration for the sparry limestone (SpaL)

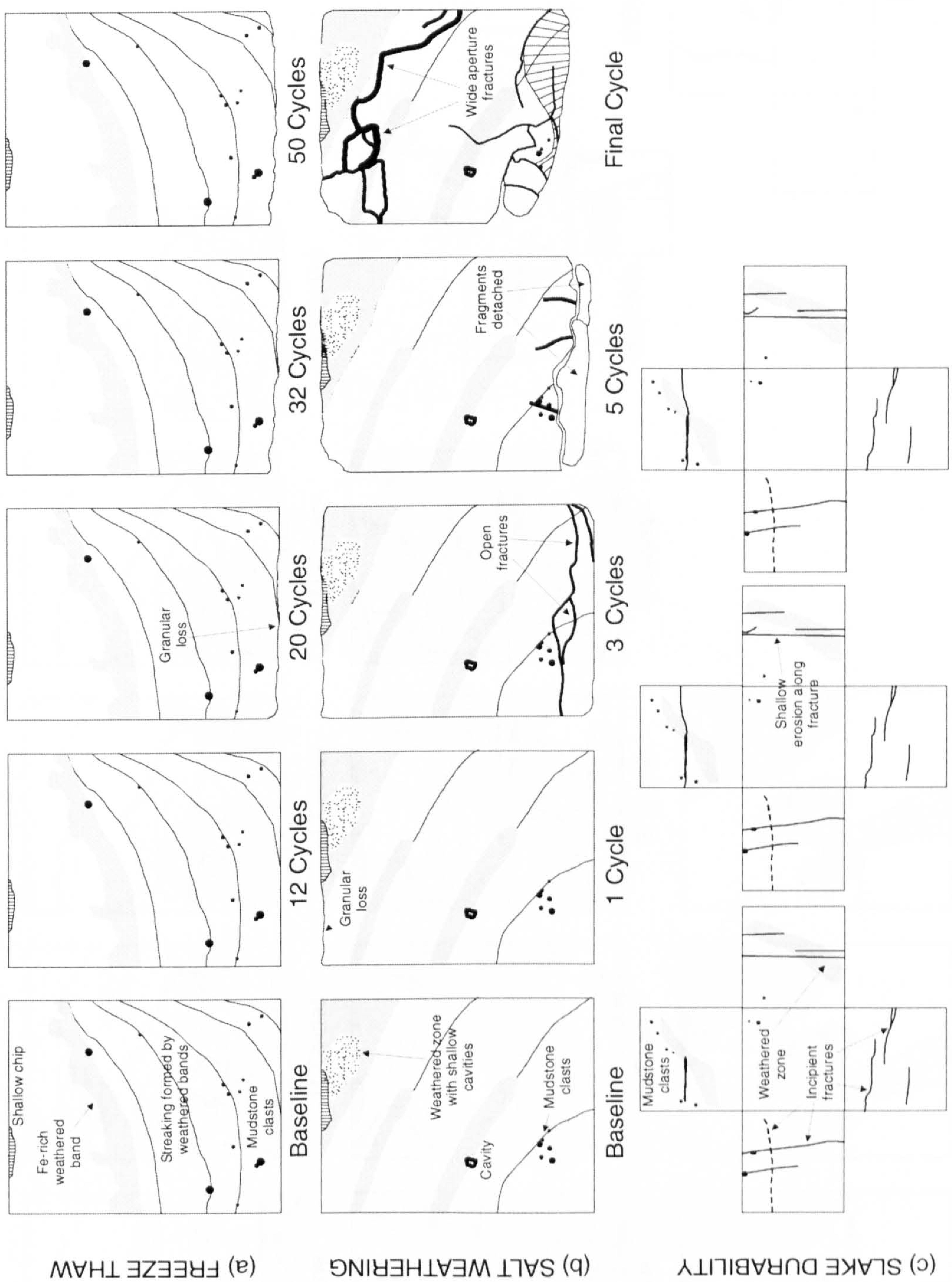


Figure 5.7 Pictorial record of deterioration for the weathered sandstone (WeaS)

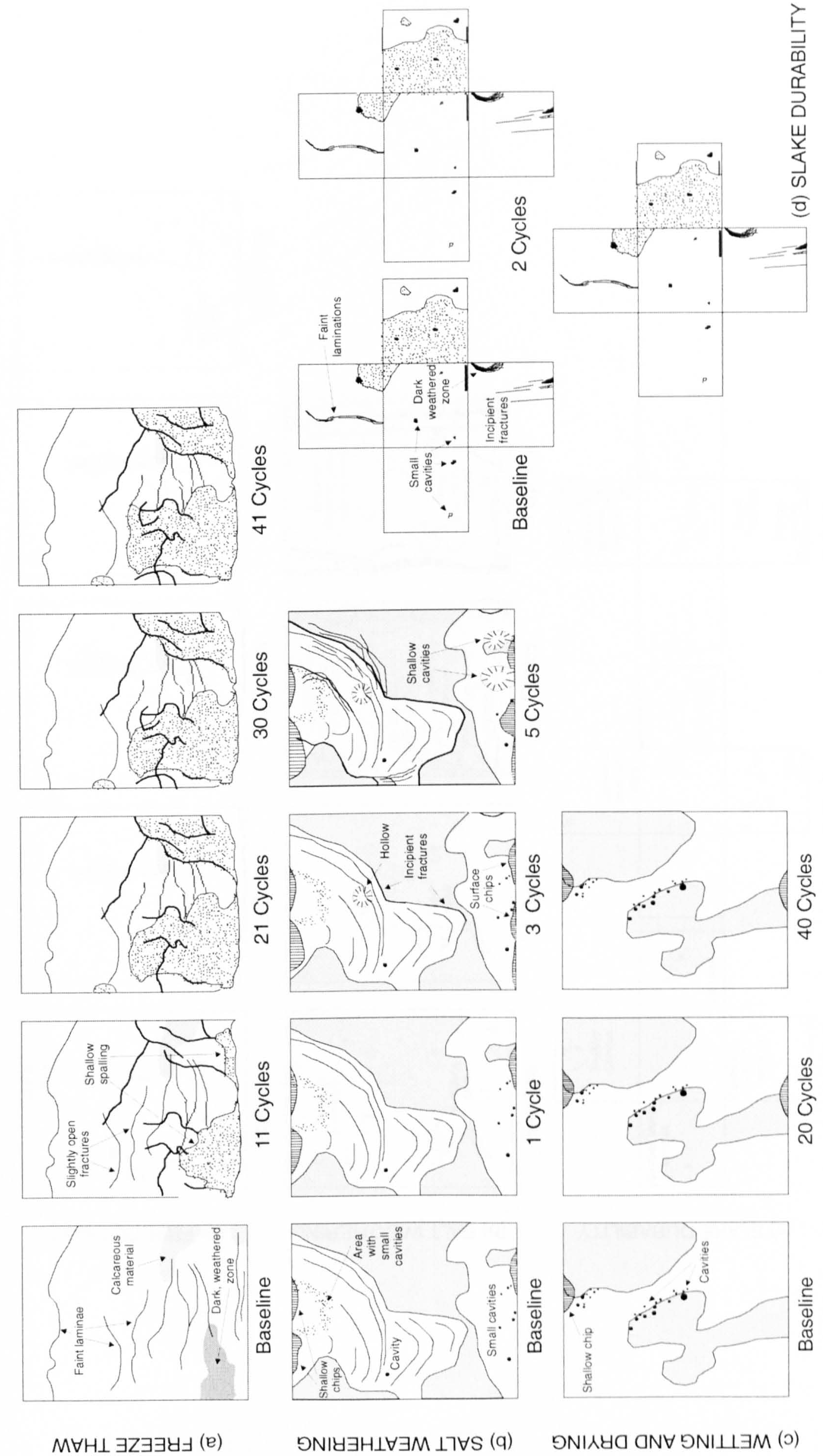


Figure 5.8 Pictorial record of deterioration for the calcareous sandstone (CalS)

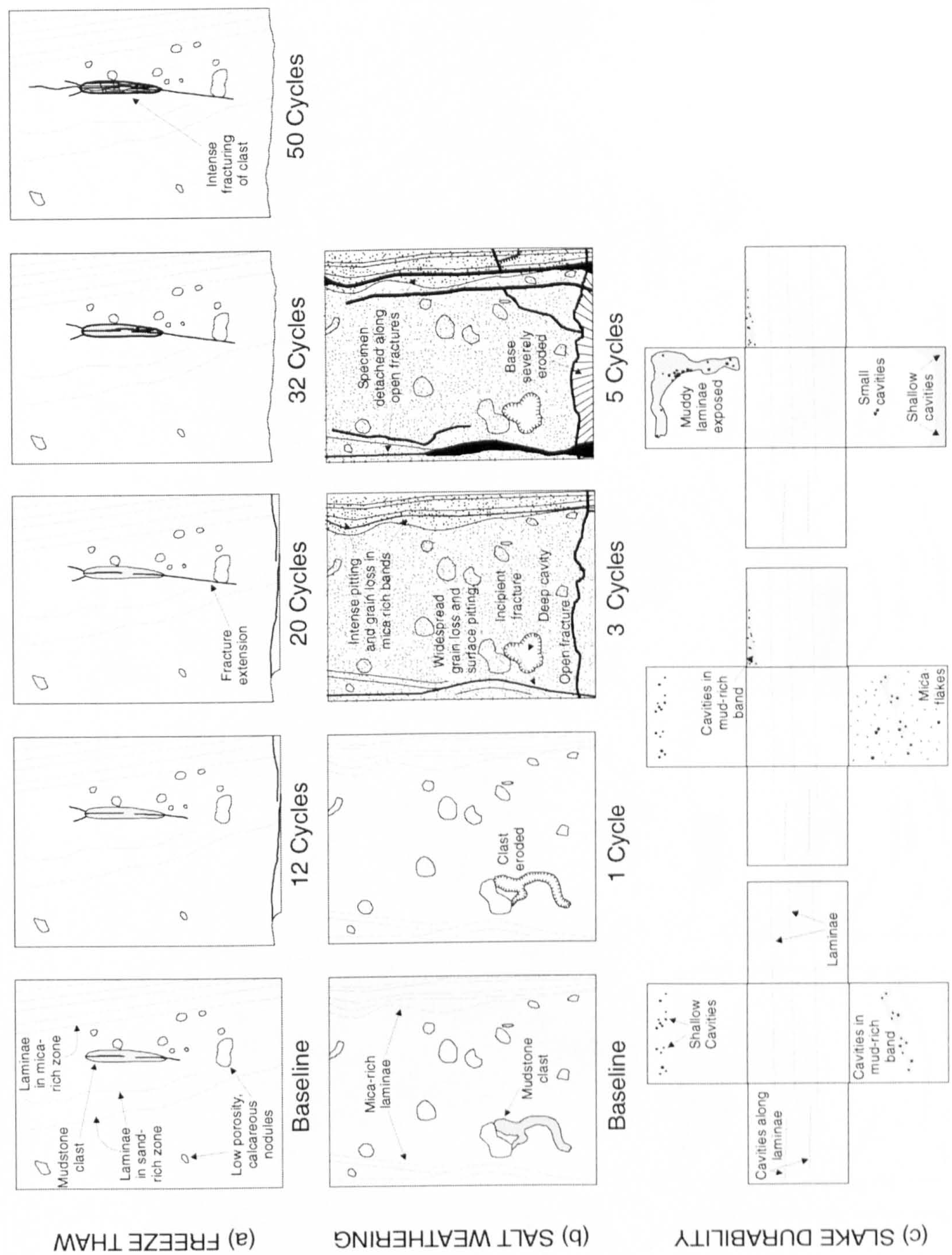


Figure 5.9 Pictorial record of deterioration for the micaceous sandstone (MicS)

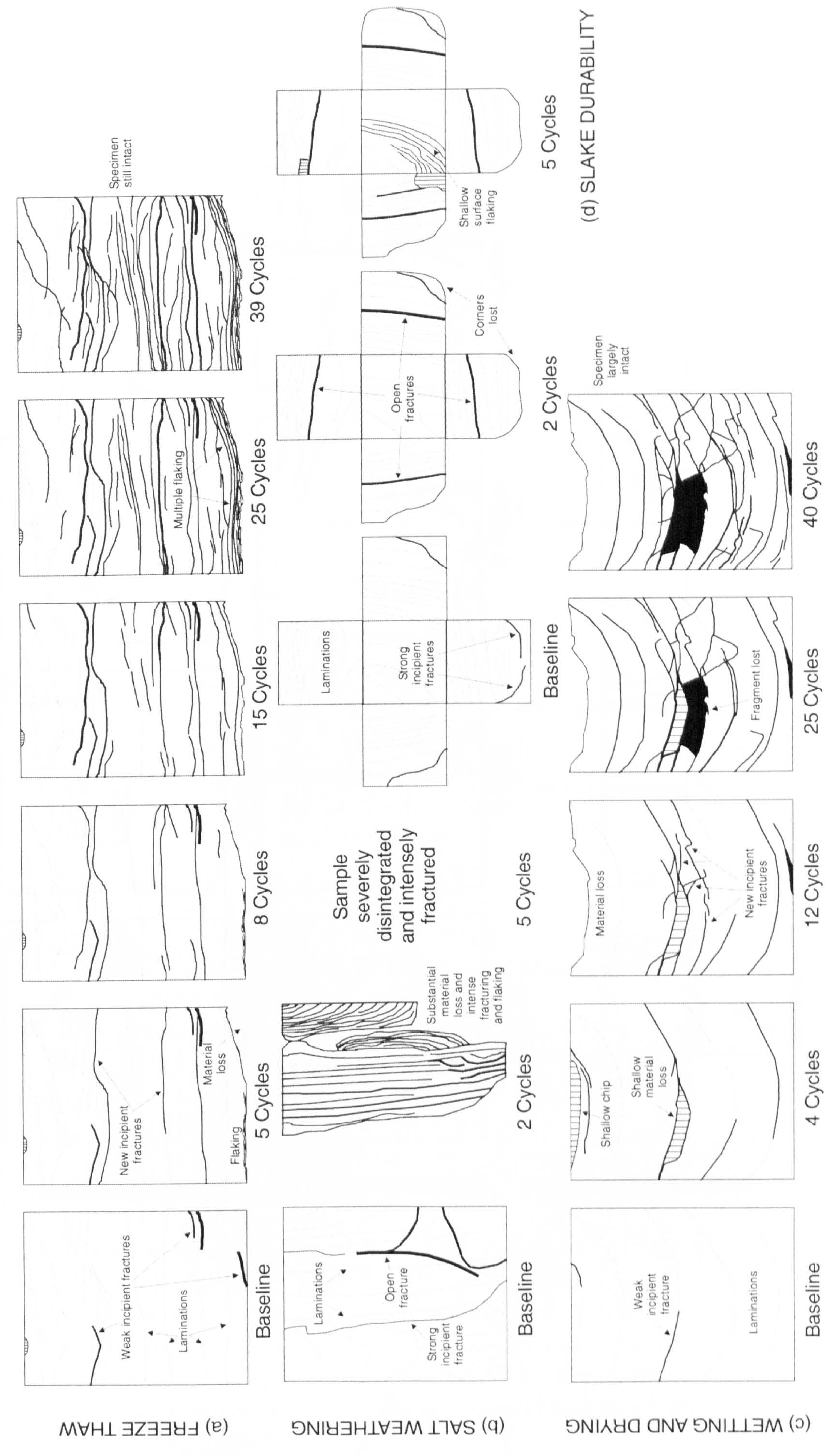


Figure 5.10 Pictorial record of deterioration for the laminated siltstone (LamZ)

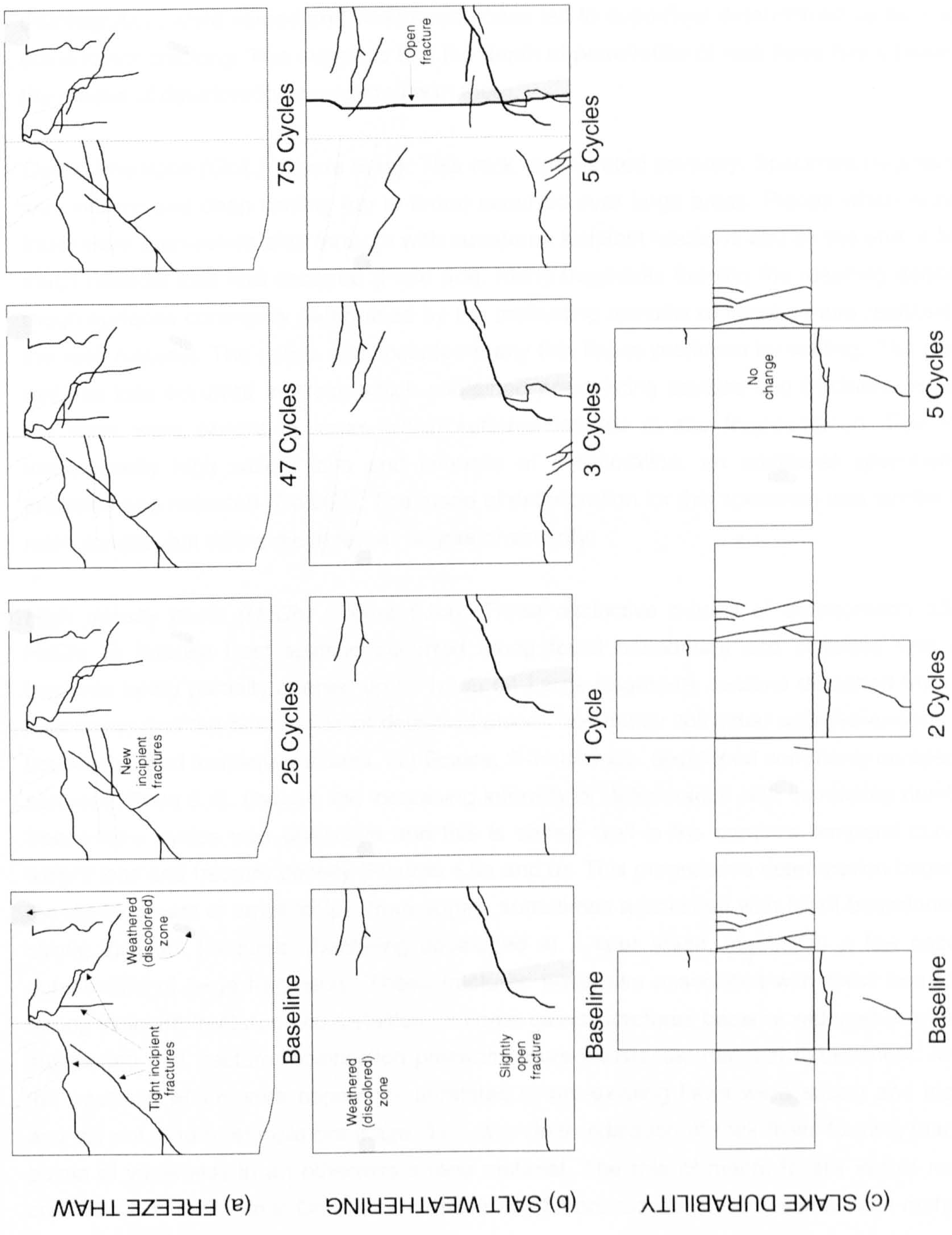


Figure 5.11 Pictorial record of deterioration for the metasediment (MetS)

Rarely, new cavities developed. (v) Localised, shallow, surface scaling occurred especially in association with voided zones (Plate 5.2). It was clear that pre-existing incipient fractures played a major role in breakdown of this rock, although some new fracturing and fragmentation were apparently unrelated to these, or to any other rock flaws. It was also notable that areas of pre-existing cracks were much more prone to penetrative breakdown by fragmentation and disintegration, while voided and weathered zones led to superficial deterioration by scaling and some minor cracking. This suggests that the depth of penetration of rock flaws has a bearing on the modes of deterioration associated with them.

Oolitic limestone (OolL) (Figure 5.4a): This rock deteriorated severely. Specimen fragmentation was intense and deep scaling (up to 8mm) occurred over large areas. Pieces which remained intact were themselves shot through with numerous incipient fractures and by the end of testing much material loss had occurred (Plate 5.3). Many fragments forming the resulting debris had rough surfaces commonly determined by the protruding remains of fossils, more resistant than the rock material. The debris also included many thin flakes produced by scaling. The greatest material loss occurred in areas which contained pre-existing cavities and fractures. However, the latter were absent in areas which suffered intense in situ fragmentation. Due to the unexpectedly high weight loss and intensity of deterioration, an additional specimen was prepared and re-tested (OolL(2)). The mode of deterioration for this specimen was similar to the main sample, but with a much lesser degree of severity.

High density chalk (HdCh) (Figure 5.5a): Three distinctive modes of deterioration affected HdCh: (i) Intense frost splitting occurred along fossil boundaries and stylolites with many fractures being partially opened up to 1mm. (ii) Large fragments became detached from most specimens and the boundaries of detached pieces commonly coincided with pre-existing fossil boundaries and incipient fractures. (iii) Scales, 3-4mm deep, developed commonly on specimen surfaces (Plate 5.4). Overall, the increasing intensity of deterioration with increasing number of freeze-thaw cycles was distinctive and this is shown well in the concave temporal curves of weight loss and fracture density (Figures 4.5a and b). This progressive deterioration began with the development of small 'chips' from edges, sometimes associated with fossil boundaries and strong incipient fractures. Fracturing developed at a later stage, leading in a few cases, to detachment of large fragments. These fractures were also associated with fossil boundaries, strong incipient fractures and stylolites. In some cases, fractures became widened or extended suggesting that fracture modification precedes more substantial material detachment. Most of the fractures which were apparently unrelated to pre-existing flaws were strong and incipient and did not lead to sample breakage. This is a clear indication of rock flaws forming planes or points of weakness in an otherwise strong material. The role of macro fossils in this respect, contrasts with their role in OolL, where they formed more resistant features in a weak material.

Sparry limestone (SpaL) (Figure 5.6a): Some specimens did not deteriorate at all, while others showed minor modification: (i) Rarely, fossils dropped out leaving a cavity behind. (ii) Small, open cracks developed at the boundaries of some cavities. (iii) Some pre-existing veins and stylolites increased in aperture and rarely specimens broke along these fractures. (iv) Rarely, new, non-persistent axial fractures developed from the top edge of specimens. Although little deterioration occurred overall, that which did was entirely dependent upon pre-existing flaws.

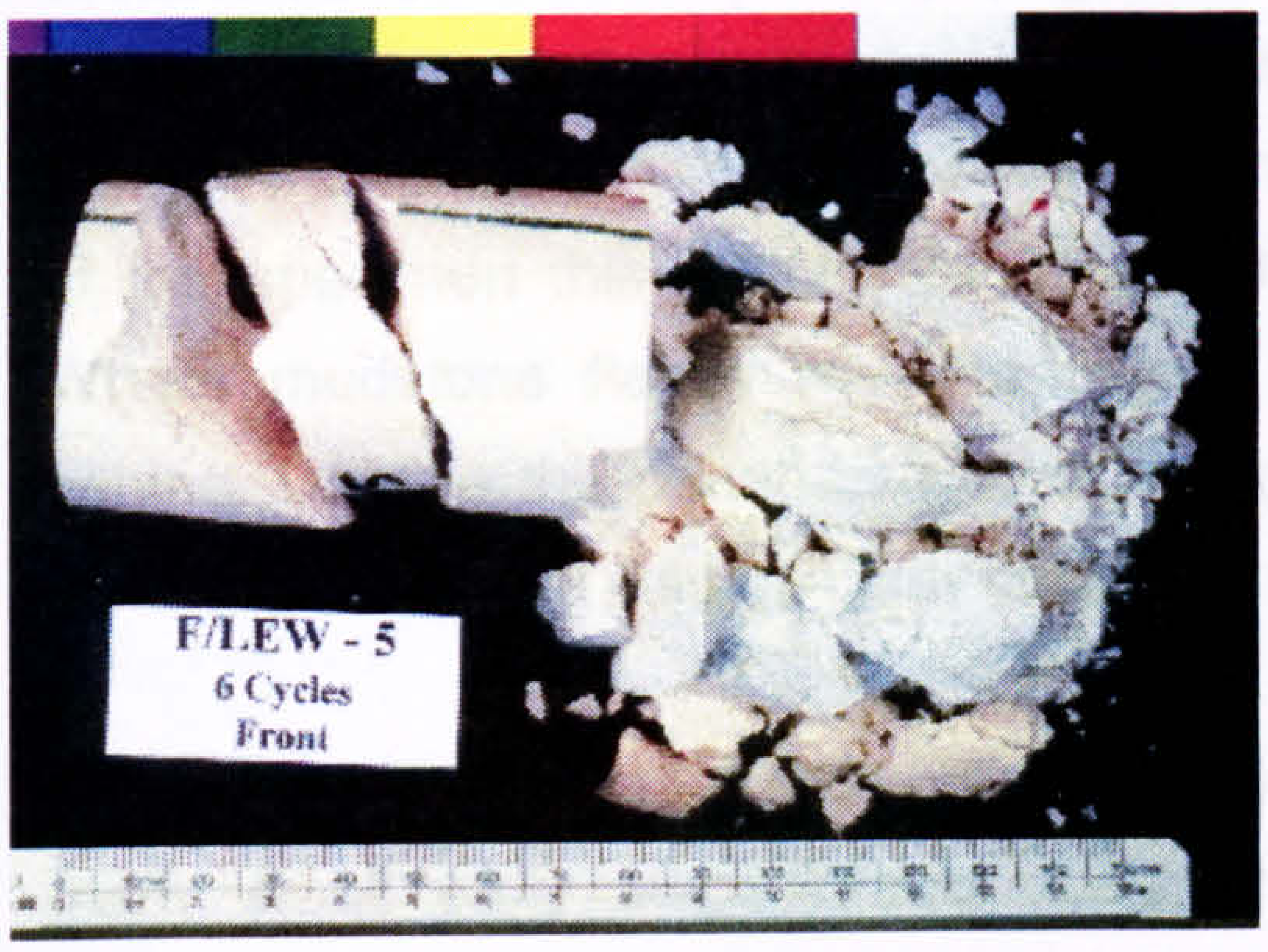


Plate 5.1 LdCh after 6 cycles

Fragmentation of this specimen resulted in generation of debris of a wide range of sizes.

Plate 5.2 MagL after 50 cycles

Parallel axial cracks can be seen extending from the missing fragment at the top of the specimen on the left hand side. The centre specimen shows shallow scaling.

Plate 5.3 OolL after 11 cycles

Deep scaling and irregular, incipient fracturing can be seen on both specimens.

Plate 5.4 HdCh after 39 cycles

Large fragments have been lost from each of these specimens, which show moderately deep surface scaling.

Plate 5.5 CalS after 41 cycles

Surface scaling and grain loss is evident on all specimens. The centre specimen shows more resistant calcite-rich areas.

Plate 5.6 LamZ after 39 cycles

These two specimens show particularly clearly the dependence of fracturing on pre-existing sedimentary structures.

Plates 5.1 to 5.6 Photographic evidence of deterioration modes for the freeze-thaw test

Weathered sandstone (WeaS) (Figure 5.7a): The principal mode of deterioration was by granular disintegration, notably along the bottom edge of specimens in association with sub-parallel fracturing. Since this basal deterioration partly corresponds with the submerged portion of the specimen this might indicate that it is a function of freezing sandstones underwater. Where mudstone flakes were removed, further grain loss occurred leaving small hollows behind. Despite the penetrative weathering bands present in this rock, it appeared to behave uniformly and there was no clear correlation between these and deterioration.

Calcareous sandstone (CalS) (Figure 5.8a): A distinct pattern of deterioration occurred in which many small early stage cracks were the focus of subsequent scaling, followed by the development of new cracks at the edges of those scaled areas, leading in turn, to their enlargement. The more rapid initial weight loss and fracture density followed by a 'tailing off' (Figures 4.8a and b) coincided with more vigorous early development of scales, with subsequent deterioration consisting of minor enlargement of those scales. In some cases, the boundary between calcareous and quartz-rich material coincided with areas of material loss due to scaling, with the more quartz-rich material being removed (Plate 5.5). Deterioration of specimen number 1 (refer to section 3.7.7) was similar to the main sample except that new cracks tended to follow the preferred orientation formed by the sedimentary fabric. There was also some grain loss on the base of this specimen and cracking and scaling were generally more intense than was typical for the main sample.

Micaceous sandstone (MicS) (Figure 5.9a): Three modes of deterioration were observed: (i) Pre-existing cracks were extended and widened. (ii) Some new fractures developed parallel to the base of specimens and also within and around large mudstone clasts. Fracturing within the clasts might have resulted from their desiccation. (iii) Limited grain loss occurred along the base of specimens and in association with mica-rich bands. Overall, deterioration was limited and was in many respects similar to that for WeaS, suggesting a textural control. The phenomenon of sub-parallel basal cracking which occurred in WeaS was also apparent here.

Laminated siltstone (LamZ) (Figure 5.10a): The extension and enlargement of pre-existing fractures parallel to the laminations, together with intense development of new fractures along laminations, dominated deterioration in this rock (Plate 5.6). Larger fractures were often linked together by smaller cross-cutting fractures and *en echelon* cracks developed in association with cross laminae, truncated surfaces and the fold hinges of deformed laminae. While some specimens remained intact, most broke into several pieces along pre-existing and new fractures parallel to laminations, which were often extremely closely spaced. The material between parallel fractures often dropped out leaving deep voids behind. Some multiple flaking occurred where laminae intersected with the ends of specimens. Deterioration was progressive in that the intensity of fractures and their apertures increased with time, and this is reflected well for some specimens in the weight loss and fracture density curves (Figure 4.10a and b).

Metasediment (MetS) (Figure 5.11a): Deterioration was minimal and mainly constituted modification of pre-existing fractures and development of new incipient fractures. After weathering, some pre-existing fractures exhibited discoloured shadow zones on either side which were not visible prior to testing. Other fractures became widened. All new cracks were angular and in some way linked or attached to other existing cracks, which were themselves, parallel to the cleavage fabric of the rock. This is another example, like HdCh, SpaL and LamZ, of a resistant material which was susceptible to weathering due to the presence of pre-existing flaws, upon which deterioration appeared to be almost entirely dependent.

5.3.2 Salt weathering

Low density chalk (LdCh) (Figure 5.2b): In the earlier stages of the salt weathering test, in situ cracking and multiple flaking occurred resulting in intensely fractured, fissile areas which coincided with discoloration and pitting evident prior to testing. Following initial resistance to fragmentation (Figure 4.2c and d), severe deterioration occurred with specimens detaching into several equi-dimensional fragments and generating substantial debris and fines. The larger fragments sometimes coincided with pre-existing flaws and contained intense fracturing and flaking (Plate 5.7). During testing it was evident that these fragments were often bound together by salt crystallised during the heating and cooling parts of the cycle, an effect reported by Booth (1990) for chalk and by McGreevy (1996) for other rocks. Detachment was able to take place only during immersion when the cohesive effect of salt was lost. Heavy pitting of all specimen surfaces occurred, leaving resistant macro fossils protruding. Scaling was also widespread.

Magnesian limestone (MagL) (Figure 5.3b): Many new axial cracks formed in association with existing cracks, but the dominant mode of deterioration was scaling, occurring in parallel, narrow strips around the circumference of specimens (Plate 5.8). There was also minor loss of material from the top and bottom of specimens, but with smaller amounts of material involved than for freeze-thaw. Some specimens developed intensely voided zones and small groups of large cavities, neither of which appeared to relate to pre-existing flaws.

Oolitic limestone (OolL) (Figure 5.4b): All specimens exhibited an initial resistance to salt weathering. Subsequently, the dominant mode of deterioration was the modification of pre-existing cavities by their enlargement, deepening, rounding and coalescence. Some minor fragmentation was also associated with these cavities. Several other mechanisms were also evident: (i) Numerous non-persistent, branching and single fractures developed parallel to the radius of specimens. Many were partially open and slight extension of specimens occurred as a result. These fractures probably relate to a hidden sedimentary fabric. (ii) Rarely, pre-existing cracks extended and coalesced, sometimes linking cavities together. (iii) Some new cavities developed. (iii) Minor grain losses occurred near the base of specimens. Variation between specimens suggests that greater resistance to deterioration was associated with an *absence* of pre-existing flaws, notably cavities, and the *presence* of a widespread cover of surface voids. Greater resistance also correlated with specimens of higher density and lower porosity.

High density chalk (HdCh) (Figure 5.5b): Heavy surface pitting occurred following salt weathering such that no original surface remained on any specimens. In addition, a dense network of angular, intersecting, incipient fractures developed by the end of testing and appeared to be independent of pre-existing flaws (Plate 5.9).

Plate 5.7 LdCh after 3 cycles (*left*)
Severe fragmentation is evident after just 3 cycles. Larger fragments show surface flaking and incipient fracturing.

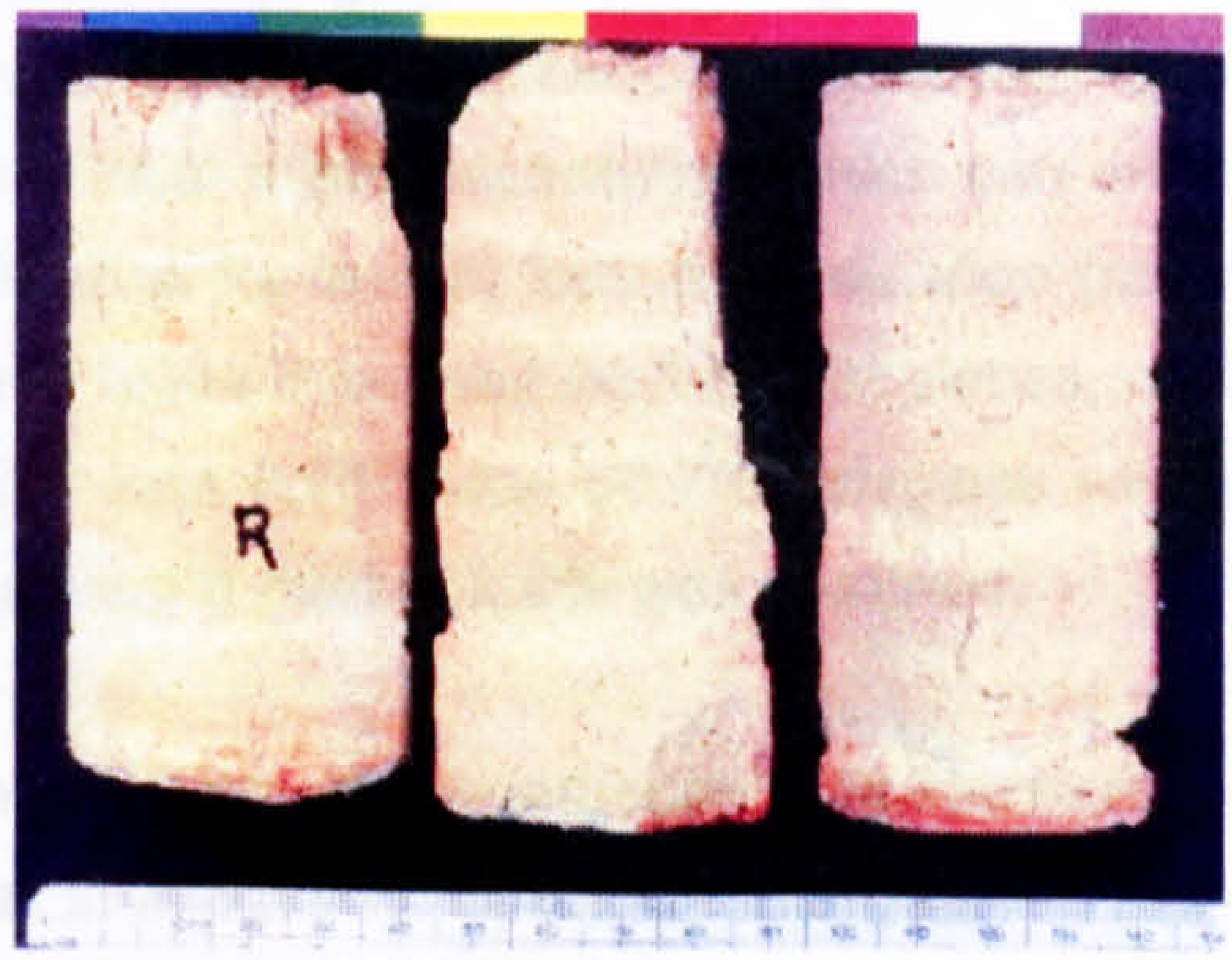


Plate 5.8 MagL after 5 cycles (*top right*)
Each specimen shows deterioration characteristic of this rock: axial cracking and radial scaling

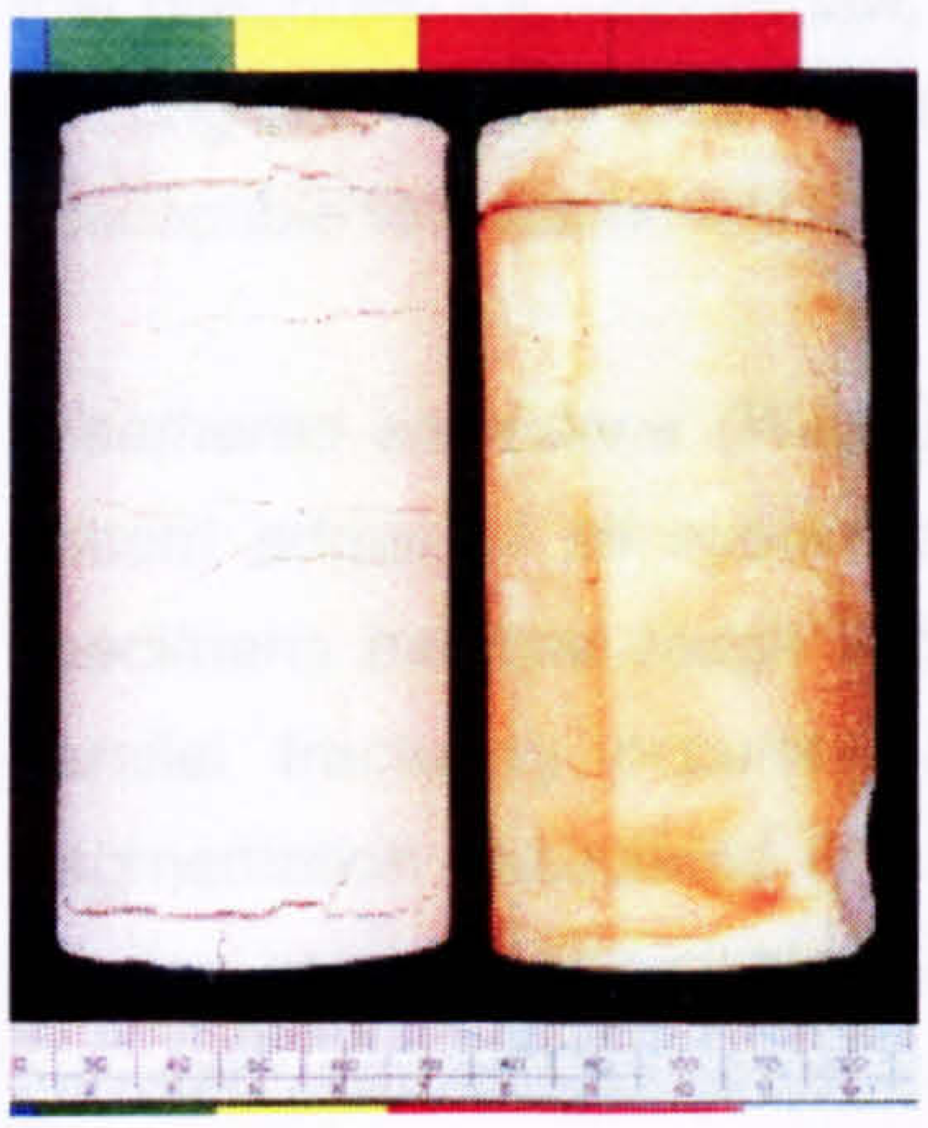


Plate 5.9 HdCh after 5 cycles (*left*)
The specimen on the left has been axially extended due to fracturing. Many narrow aperture incipient fractures are not visible in the photograph. Discoloration of the right specimen is unrelated to deterioration.

Plate 5.10 SpaL after 3 cycles (*right*)
Tight incipient fractures are picked out by moisture. A fragment broke at the top from a pre-weathered fracture.

Plate 5.11 WeaS after 6 cycles (*above*)
These specimens show substantial surface grain loss and pitting, as well as wide aperture fractures unrelated to pre-existing flaws.

Plate 5.12 MicS after 5 cycles (*above*)
These show fracturing; surface pitting; calcite nodules protruding; variations in grain loss with both grain size and along laminations.

Plate 5.13 LamZ after 2 cycles (*left*)
Extensive fracturing after just two cycles.

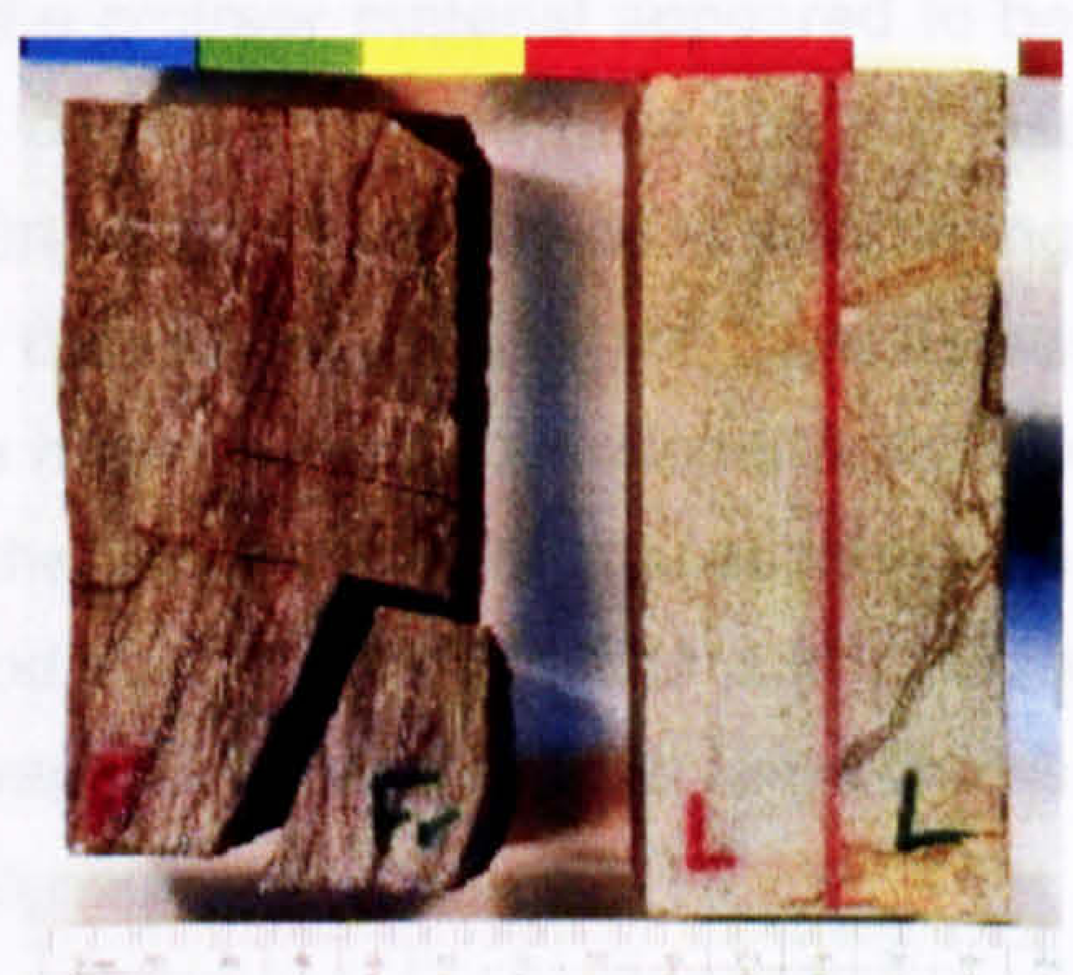


Plate 5.14 MetS after 5 cycles (*right*)
Development of incipient and open fractures, generally associated with cleavage.

Plates 5.7 to 5.14 Photographic evidence of deterioration modes for the salt weathering test

Many of these fractures had apertures up to 1.5mm leading to axial extension of some specimens. It was also common for one or two small fragments to become detached. One exception to this provides a good illustration of the potential misrepresentation which can arise by using the largest remaining piece (LRP) as the criterion for weight loss determination (refer to discussion in section 3.4.1). This particular specimen broke into three equi-length pieces, one of the fractures forming along a stylolite seam. Weight loss (LRP) was 60.7%, whereas when determined by the criterion adopted in this research, a weight *gain* of 2.2% was obtained.

Sparry limestone (SpaL) (Figure 5.6b): Deterioration following salt weathering was identical to that described for freeze-thaw, with the exception that in addition to aperture enlargement, pre-existing cracks also extended (Plate 5.10). It seems likely that these fractures would have been susceptible to detachment had further cycles of salt weathering been conducted.

Weathered sandstone (WeaS) (Figure 5.7b): Substantial grain loss occurred on the top and bottom edges of all specimens leading to significant rounding. The small scale texture of specimens became rough and pitted with no specimens retaining any original surface. Sub-parallel fracturing occurred along the bottom edge with further grain loss and minor fragmentation associated (Plate 5.11). Other new irregular cracks also formed, occasionally relating to mudstone clasts. As with the freeze-thaw test, there was no apparent relationship between material loss or crack development and the location of pre-existing weathered bands.

Calcareous sandstone (CalS) (Figure 5.8b): Deterioration was somewhat dependent on the proportion of calcite and quartz present: (i) Most quartz-rich surfaces became roughened and slightly pitted, but calcareous areas remained smooth and were much less affected. This is similar to the findings of the freeze-thaw test in which it was largely the quartz-rich areas from which material was removed. (ii) The *boundaries* of calcite and quartz-rich material were hollowed out and associated minor scaling and fracturing occurred. The only other deterioration observed was the occasional development of small cracks at the top and bottom of specimens, generally parallel to laminations. These cracks were generally isolated and tightly closed.

Micaceous sandstone (MicS) (Figure 5.9b): Most surfaces became rough and pitted with widespread grain loss occurring at the base of specimens and very little original surface being retained. Coarser, quartz-rich material suffered less grain loss than the finer, mica-rich material. This is converse to the effect of slaking on WeaS, where the coarser material appeared to be more susceptible to breakdown. The white, pale-coloured patches referred to in the pre-test description usually stood proud at the end of testing, indicating greater resistance to weathering than the host material. Considerable scaling also occurred, occasionally leading to large flakes being lifted from the surface, sometimes coincident with the hinges of folded laminae. A variety of new fractures developed: (i) Small, isolated cracks near the base or middle of specimens. (ii) Very large, open cracks parallel to laminae, located at boundaries between fine and coarse sandstone, and sometimes leading to breakage and minor fragmentation. (iii) Other large cracks with moderate apertures, commonly leading to breakage and scaling (Plate 5.12).

Laminated siltstone (LamZ) (Figure 5.10b): Deterioration due to salt weathering was similar in style to that for the freeze-thaw test, but much more intense (Plate 5.13). Extension of pre-existing cracks and intense development of new cracks rendered specimens extremely weak. Considerable material loss occurred, with specimens barely resembling their original cylindrical form by the end of testing.

Metasediment (MetS) (Figure 5.11b): Many new, faint incipient fractures developed. Some were strong while others were weak with a wide aperture. Most new cracks were linked to each other and to pre-existing cracks as was the case for freeze-thaw. One specimen suffered minor material loss (Plate 5.14).

5.3.3 Wetting and drying

Low density chalk (LdCh) (Figure 5.2c): Minimal deterioration occurred as a result of wetting and drying, consisting of the slight enlargement of cavities around fossils and rare development of new closed fractures.

High density chalk (HdCh) (Figure 5.5c): With the exception of small surface chips occurring on the base of some specimens, possibly induced by handling, there was no visible deterioration of specimens due to wetting and drying.

Calcareous sandstone (CalS) (Figure 5.8c): The only visible deterioration was some small handling-induced chips.

Laminated siltstone (LamZ) (Figure 5.10c): Deterioration due to wetting and drying was very similar in mode and severity to that for freeze-thaw (Plate 5.15).

5.3.4 Slake durability

Low density chalk (LdCh) (Figure 5.2d): Substantial rounding of specimens occurred, leaving some specimens almost ball-like in appearance by the end of testing (Plate 5.16). The texture of the resulting surfaces was rough and pitted. A cycle of deterioration became apparent in which abrasion of the surface revealed previously hidden fossils (and weathered zones) which subsequently dropped out due to further grain loss. The resulting cavities were then smoothed and planed by the continuing abrasion.

Magnesian limestone (MagL) (Figure 5.3c): New, incipient discontinuities occasionally developed and pre-existing cracks extended, but the main form of deterioration was the development of new voids and larger, deeper cavities, particularly in the more pre-weathered parts of the rock. There was also minor rounding and smoothing of corners and edges, and very minor material loss.

Oolitic limestone (OolL) (Figure 5.4c): Grain loss, resulting in a rough, irregular surface occurred in most specimens. While overall rounding of specimens did not occur, pre-existing cavities, together with new voids created by fossils which had fallen out, became rounded and less distinct (Plate 5.17). The greatest deterioration occurred in specimens with many pre-existing cavities while a widespread cover of shallow surface voids appeared to be associated with greater resistance to deterioration, as was the case with the salt weathering test.

High density chalk (HdCh) (Figure 5.5d): Very slight pitting occurred at edges and corners.

Plate 5.15 LamZ after 40 cycles of wetting and drying (*above*)

Typical deterioration for LamZ, with many lamination-parallel fractures. Some linking and *en echelon* fractures can also be seen.

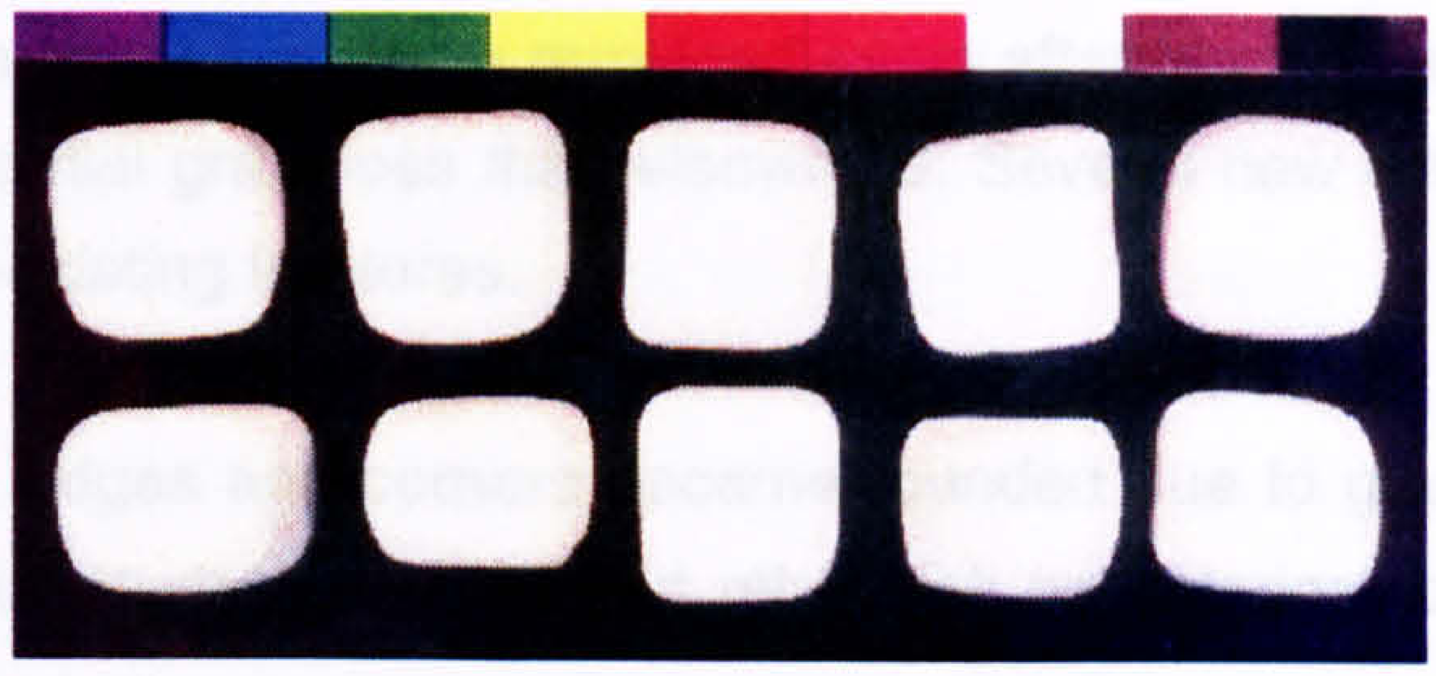


Plate 5.16 LdCh after 5 cycles of slaking (*above*)

Intense rounding of corners and edges. No original surfaces remain.

Plate 5.17 OolL after 5 cycles of slaking (*left*)

Some original surfaces remain, but moderate rounding of corners and edges has occurred. Cavities in some specimens have been enlarged by rounding and coalescence.

Plate 5.18 WeaS after 5 cycles of slaking (*left*)

These specimens show general rounding of corners and edges.

Plate 5.19 LamZ after 5 cycles of slaking (*right*)

Although rounding of corners and edges has occurred due to abrasion, most specimens also broke along laminations. Some incipient fractures remained but in most cases, breakage occurred along them.

Plates 5.15 to 5.19 Photographic evidence of deterioration modes for the wetting and drying and slake durability tests

Sparry limestone (SpaL) (Figure 5.6c): There was no visible sign of deterioration due to slaking, although minor weight loss was recorded, probably associated with minor flaking around individual crystals.

Weathered sandstone (WeaS) (Figure 5.7c): Most specimens became rounded due to grain loss, which was slightly greater in coarser material (Plate 5.18). The least resistant specimens contained many pre-existing, open cracks and although these remained intact after testing they were, nevertheless, the focus of more substantial grain loss than elsewhere. Several new sub-parallel cracks developed in parallel with pre-existing fractures.

Calcareous sandstone (CalS) (Figure 5.8d): Edges and corners became rounded due to grain loss. The aperture of one pre-existing crack became enlarged, but otherwise no deterioration was observed.

Micaceous sandstone (MicS) (Figure 5.9c): General smoothing of corners and edges occurred due to grain loss, which was greatest in mica-rich layers. Mica-rich layers were also the focus of the many new, small voids which developed.

Laminated siltstone (LamZ) (Figure 5.10d): Smoothing of edges and corners occurred and pre-existing cracks were extended and opened. Some new, lamination-parallel incipient fractures also developed and some breakage and fragmentation of specimens occurred around pre-existing cracks (Plate 5.19).

Metasediment (MetS) (Figure 5.11c): Very minor rounding of edges and corners occurred but was barely discernible. Some pre-existing cracks were also extended and there was rare flaking. Some specimens did not show any change at all.

5.3.5 Summary of deterioration modes

From the above descriptions of sample deterioration, it is apparent that a number of distinctive deterioration modes can be identified, and these are summarised below.

5.3.5.1 Rapid, severe disintegration

Severe disintegration describes major breakdown of a rock such that high weight losses are recorded and a considerable quantity of debris and fines is generated. This occurred in the weakest rocks (LdCh and OolL), but also in LamZ by virtue of its sedimentary structure.

5.3.5.2 Incipient fracturing and multiple flaking

Incipient fractures are mechanical breaks in rock which retain some tensile strength (see section 2.4.1 of Chapter Two). Fracturing occurred in all samples following freeze-thaw and salt weathering. However, it featured particularly strongly in LamZ and MagL for all tests, with multiple flaking also occurring in the former due to the very close spacing of laminae. This is comparable to the results of an experimental frost weathering study by Brockie (1972) in which several highly resistant schists broke along pre-existing "fissures or lines of weakness" early in the test procedure. Fracturing also occurred at a low density in SpaL and MetS, two rocks which otherwise resisted deterioration due to weathering.

5.3.5.3 Breakage

Detachment of relatively large fragments along weak fractures can be described as breakage. In rocks which were affected predominantly by breakage, fracture density tended be low. Weight loss was also usually low because the detached fragments were few in number, and mostly exceeded 10% of the initial specimen dry mass. Breakage generally characterised the stronger rocks (HdCh, SpaL, WeaS, MicS) which broke along isolated, pre-existing flaws. Breakage also occurred in LamZ, but this sample was distinctive in that each 'fragment' was itself intensely dissected by incipient fractures and multiple flaking. Breakage did not occur commonly due to slaking. Minor fragmentation (see 5.3.5.4 below) appeared to be less common for samples which experienced breakage.

5.3.5.4 Minor, and local fragmentation

The loss of small rock fragments, usually from a localised area and often associated with minor cracking, characterises this deterioration mode. Minor fragmentation occurred in a number of samples but notably those which resisted severe disintegration, breakage and fracturing (eg MagL, SpaL, WeaS). Minor fragmentation often developed following salt weathering.

5.3.5.5 Scaling

The peeling off of surface layers of rock is known as scaling (and 'contour scaling', 'exfoliation' and 'spalling'). Smith and McGreevy (1983) showed that the penetration of scales due to salt weathering related to sub-surface crystallisation of salts at the corresponding frequent wetting depth. In a similar way, scaling due to freeze-thaw could reflect penetration of the migrating ice front into the rock (Lienhart 1988). Scaling varies from localised peeling to persistent bands encircling the specimen, with the depth of scales varying from around 1 to 8mm. Scaling did not feature in the wetting and drying or slaking tests, but occurred commonly due to freeze-thaw, being present in 50% of the rocks tested, including four out of five of the limestones.

5.3.5.6 Granular loss

Granular loss usually occurs due to weakening of cement bonds between grains, allowing them to drop out, or if subject to abrasion and impact, to be forced out. This mode of deterioration was, unsurprisingly, notably more common in the granular rocks (OolL, WeaS, CalS, MicS, LamZ) especially those of coarser texture. The close association between sandstone texture and granular disintegration has been identified by other workers (Smith at al 1994; Robinson and Williams 1996), indication of a strong textural influence on deterioration mode in this case. It was common for samples which suffered grain loss also to suffer minor fragmentation. It is likely that other modes of deterioration would be superimposed on grain loss if suitable flaws were present, or perhaps if environmental conditions were more severe or prolonged. This suggestion is supported by the cracking associated with mudstone clasts which occurred in MicS, and is also supported by an experimental salt weathering study by Smith and McGreevy (1983). In this, the progressive nature of breakdown was noted, in which initial grain loss was succeeded by cracking, which subsequently developed into flaking. Granular loss was very common following slaking, occurring in seven out of ten samples. This probably reflects the abrasive nature of the test which tends to promote granular breakdown.

5.3.5.7 Surface pitting and cavity development

Surface pitting only occurred as a result of salt weathering (LdCh, HdCh, CalS, MicS), notably in the finer grained rocks. Cavity development only occurred in two samples, in MagL after every test, and in OolL following salt weathering.

5.3.6 The influence of weathering process on deterioration mode

It is clear from an analysis of deterioration modes that the rocks tested here can be divided into two groups: those which showed a similar deterioration mode regardless of the weathering conditions, and those for which deterioration mode varied according to the weathering conditions. The former group is indicative of environmental control being subservient to rock control in determining deterioration mode (Yatsu 1966). The rock properties which constitute this rock control, however, are not the same in each case. Deterioration of LamZ, for instance, was dominated by intense fracturing along laminations, a reflection of the influence of structural control. SpaL and MetS, however, generally resisted deterioration because of their intrinsic mechanical strength. The common occurrence of granular loss and minor fragmentation in the sandstones (WeaS, CalS, MicS) is evidence that in this case, deterioration mode is more strongly influenced by rock texture. It is likely for each of these cases that although certain rock properties might have a determining influence on deterioration mode, a combination of rock controls will act. This is certainly the case for MagL where development of cavities following most weathering tests was influenced by pre-existing flaws, but where an enhanced resistance to weathering generally, probably relates to its interlocking texture.

Three rocks (LdCh, OolL, HdCh) showed a contrasting deterioration mode for each test, indicating that environmental control overrides the influence of rock properties. HdCh, for instance, largely resisted deterioration due to wetting and drying, experienced minor grain loss following slaking, suffered intense, incipient fracturing and surface pitting due to salt weathering and deep scaling and breakage due to freeze-thaw. It was notable that in the discussion of the role of weathering processes in the severity of deterioration (section 4.2.7.1 in Chapter Four), OolL and HdCh were categorised as rocks which showed a variable resistance to the weathering tests conducted. For these, therefore, the nature of the environmental regime would appear to have a major influence on both the severity and the mode of deterioration. In the case of LdCh, which deteriorated severely for all tests except wetting and drying, the influence of environmental regime has more influence on mode of deterioration than severity. It is also clear that the nature of the weathering process might cause rocks to deteriorate in a certain way. The slaking test in particular, engendered a distinctive deterioration response for most rocks.

Namely, granular loss was overwhelmingly the most common deterioration mode, regardless of rock type.

5.4 Role of Pre-Existing Flaws in Rock Deterioration

The aim of this section is to consider the influence of rock flaws on the mode and severity of deterioration. The coupled relationship between rock flaws and their host lithology is also evaluated, in other words, do pre-existing flaws exert their influence in a way which is dependent on the characteristics of the rock?

For every sample tested here, at least some deterioration coincided with one or more types of rock flaw, although this was less evident for slake durability. In some cases, the presence of pre-existing flaws appeared to be incidental in that deterioration would have occurred even in their absence. The relationship between deterioration and pre-existing flaws can be best evaluated with respect to linear, point and dispersed types of flaw:

5.4.1 Linear flaws

Of all pre-existing flaws observed (see Figure 5.1), deterioration was most commonly associated with linear weaknesses, particularly open, weak or strong incipient fractures (Fo, Fw, Fs) and, to a lesser extent, stylolites, veins and laminations. Deterioration was more likely to be associated with laminations when they were very closely spaced, though this distinction was less obvious in the salt weathering test. The importance of linear weaknesses in providing a focus for deterioration cannot be understated because in every sample containing such features (particularly Fo and Fw), there was at least some deterioration associated with them. This is not strictly true for LdCh following freeze-thaw because it was difficult to verify the relationship between pre-existing flaws and the severe deterioration which occurred. Linear weaknesses did not always break apart due to weathering, but commonly became extended or widened. The development of new, sub-parallel sets of cracks alongside pre-existing fractures was very common and small scale fragmentation along linear weaknesses also occurred.

5.4.2 Point flaws

Point sources of weakness include macro fossils and shell fragments, lithic clasts and nodules. Their influence on deterioration was determined by their strength relative to the host rock. For instance, relatively weak mudstone clasts in stronger rock (WeaS, MicS) were the focus of minor fragmentation, granular loss and cracking. Similarly, where large fossils were present in strong rocks (HdCh, SpaL) they formed weaknesses and were the source of minor fragmentation, cracking and scaling. Conversely, strong calcite nodules (MicS) resisted deterioration and protruded from the surface at the end of testing due to grain loss from around the nodules in the weaker host material. Similarly, where fossils were present in weaker rocks (LdCh, OolL), the host material deteriorated leaving the more resistance fossils protruding from the surface and creating an irregular surface texture. Macro fossils (Om) were second only to linear weaknesses in their close association with deterioration.

5.4.3 Dispersed flaws or lithological heterogeneities

Variations in grain size, porosity and mineral composition often coincided with variations in grain loss and minor fragmentation, and it was common for localised scaling and cracking to occur at the boundaries of these lithological variations. Cavities and voids were also the focus of small scale deterioration processes, notably very localised granular loss, minor fragmentation, cracking, and the enlargement, rounding and coalescence of cavities. Some specimens containing large areas of many small voids (MagL and OolL) showed greater *resistance* to deterioration and these could relate to the effect of pore structure on fluid absorption (see discussion in 4.4.5). Some of the larger, isolated cavities behaved more as point sources of weakness and were the focus of considerable modification and cracking.

There is some indication from these results that the degree of penetration and persistence of pre-existing flaws influence their role in deterioration. Non-penetrative, non-persistent flaws such as discoloured spots and shallow cavities were more likely to be associated with minor and localised deterioration (eg by grain loss or scaling), whereas persistent incipient fractures were more likely to be associated with deep scaling, large scale fragmentation and fracturing. It is also notable that although macro fossils had a significant role in deterioration, the same is not true for much smaller and therefore less penetrative shell fragments. Other discoloration features such as streaks, bands and patches, although persistent in many cases, did not appear to have an influence on deterioration. This was probably because there was no actual weakening of the rock material involved.

There were some pre-existing flaws, such as shear and deformation structures, which appeared to have little influence on deterioration, although the hinge areas of folded laminae sometimes coincided with minor fragmentation and cracking. Conversely, there were many examples of deterioration which were apparently unrelated to pre-existing flaws. In some cases, this apparent lack of influence of flaws might simply reflect the fact that flaws were not visible in hand specimen.

5.4.4 Discussion

Most rocks which were consistently resistant or consistently susceptible to all weathering tests also exhibited a very similar mode of deterioration for each test. Conversely, two rocks (OolL, HdCh) which showed very different resistance to each test also responded variably in terms of deterioration mode. Several simplified models are proposed which attempt to relate the response of rocks to weathering in terms of rock and environmental controls:

Model 1: Rock properties are subservient to environmental conditions in controlling the severity and mode of deterioration due to weathering. OolL and HdCh characterise this model.

Model 2: Environmental conditions are subservient to rock properties in controlling the severity and mode of deterioration due to weathering. This group can be sub-divided with respect to certain rock properties (eg rock strength, texture and structure):

Model 2A: The intrinsic low mechanical strength of weak rocks usually equates with high porosity (Winkler 1994) and hence greater susceptibility to many weathering processes (eg McGreevy 1982 and herein). Since the absorption of moisture necessary for processes such as freeze-thaw, salt weathering and wetting and drying depends upon rock microstructure, it is likely that deterioration in these rocks will be more closely associated with void-dependent properties and microcracks than with macro-flaws. The role of pre-existing flaws, therefore, is largely incidental to any deterioration that occurs. LdCh characterises this model.

Model 2B: The intrinsic high mechanical strength of strong rocks contrasts with the fundamental weakness provided by any pre-existing flaws present, resulting locally in regions of low tensile strength. Since the usual corollary to high mechanical strength is low porosity (McGreevy 1982; Winkler 1994), flaws also provide preferential routes for moisture ingress (Nieminen and Uusinoka 1988). This moisture is essential for many weathering processes, including its potential role in the swelling of clay minerals (McGreevy 1982) and stress corrosion (Whalley et al 1982). Flaws are thus exploited and are the focus of most deterioration taking place, which might, nevertheless, be minimal. Where flaws are present in abundance, deterioration also has the potential to be severe. SpaL, LamZ and MetS characterise this model.

Model 2C: Textural properties might predispose sandstones to deteriorate predominantly by granular loss (eg Robinson and Williams 1996). This might be because of the relative ease with which microcracks can develop in intergranular cement where this is weaker than the constituent grains. Propagation of grain boundary microcracks might also be frequently be halted in coarse-grained rocks so that long cracks often do not develop. The result is that disintegration occurs at the scale of the grains rather than through the rock material as a whole. MagL, WeaS, CalS and MicS characterise this model sub-group.

Clearly, it is also possible to envisage rocks whose behaviour would be transitional between any of these models, and models 2A and 2B in particular.

It seems that pre-existing flaws are particularly important in deterioration of stronger rocks and their direct influence diminishes in weaker rocks as the influence of other rock properties and environmental factors increases. For strong rocks, these findings support the conclusions of Tharp (1987) and Douglas (1981) that environmental conditions are subordinate to discontinuities in terms of their effect on weathering. For weak rocks, the findings indicate that macro-flaws are less important in weathering susceptibility than other rock properties such as mechanical strength and texture. For some rocks there are also indications that environmental conditions have a much greater influence in determining the mode of breakdown. Broad relationships between deterioration mode and pre-existing flaws have also been identified.

5.5 Modifications to Rock Properties During Weathering

As seen in Chapter Four, the role of rock properties in determining susceptibility to weathering is difficult to determine with any precision. This is partly because there is no single property which dominates rock behaviour, but rather a combination of properties (Matsuoka 1990a). It is also partly because the *mechanisms* by which weathering leads to rock breakdown are not clearly understood. Indeed, interactions between lithological, mechanical and pore properties of rocks and the nature of weathering processes affecting them, makes for a very complex system. In the second part of this chapter, consideration is given to the way in which certain pore-dependent and mechanical properties are *modified* as a result of weathering. The aim is to determine what these modifications imply about the breakdown mechanisms taking place.

Although there is an expanding literature dealing with changes to mechanical properties (eg sonic velocity and elasticity) due to weathering (eg Allison 1988, 1990; Goudie et al 1992; Allison and Goudie 1994; Remy et al 1994; Murphy and Inkpen 1996; Allison and Bristow 1999), there are few studies in which measurements of modifications to void-dependent properties such as porosity, pore size distribution, degree of saturation and water absorption capacity are presented (eg Brockie 1972; Accardo et al 1981; Fitzner 1988; Tugrul 1997; Pombo-Fernandez 1999). There are several acknowledgements in the literature that modifications to pore properties arising from weathering occur. For instance, Bland and Rolls (1998) refer to experimental work (no reference cited) in which repeated wetting and drying resulted in increases in sample saturation and the sizes of pores and microcracks. Whalley et al (1982) refer to the phenomenon of continually changing capillarity within a rock fracture as weathering proceeds. However, there are relatively few experimental data presented in the literature which support and verify these assertions.

As described in Chapter Three, a range of rock properties have been measured, either on a before and after weathering basis, or at several intervals during weathering. The properties measured include effective porosity, dry density, porosity and density as determined from mercury intrusion porosimetry, pore size distribution, microporosity, mechanical strength (freeze-thaw only) and elasticity. The results of modifications to these properties due to weathering are now presented and discussed.

5.5.1 Effective porosity (n_e)

Percentage change in effective porosity for individual specimens for each weathering test is shown in Figures 5.12, 5.13, 5.14 and 5.15. Absolute pre- and post-test values are summarised in Table 5.2 (page 150).

5.5.1.1 Freeze-thaw (Figure 5.12)

Freeze-thaw produced an increase in mean sample n_e for MagL, HdCh and LamZ (Figure 5.12b, d and i). LdCh showed an overall reduction (Figure 5.12a) but there was considerable variability between specimens. Mean change for the remaining rocks was negligible or insignificant. The absence of any corresponding change in density for MagL and HdCh (Table 5.2) indicates that there was no change in total pore volume, but that the increase in n_e represents improved saturation. That there was a reduction in ρ for LamZ suggests that in addition to any change in degree of saturation which occurred, there was also a net increase in total pore volume for this rock. The largest absolute increase in n_e of 3% was observed in HdCh. For these three rocks (MagL, HdCh and LamZ) there was much greater consistency between individual specimens than for other rocks (Figures 5.12b, d and i). Some rocks also showed

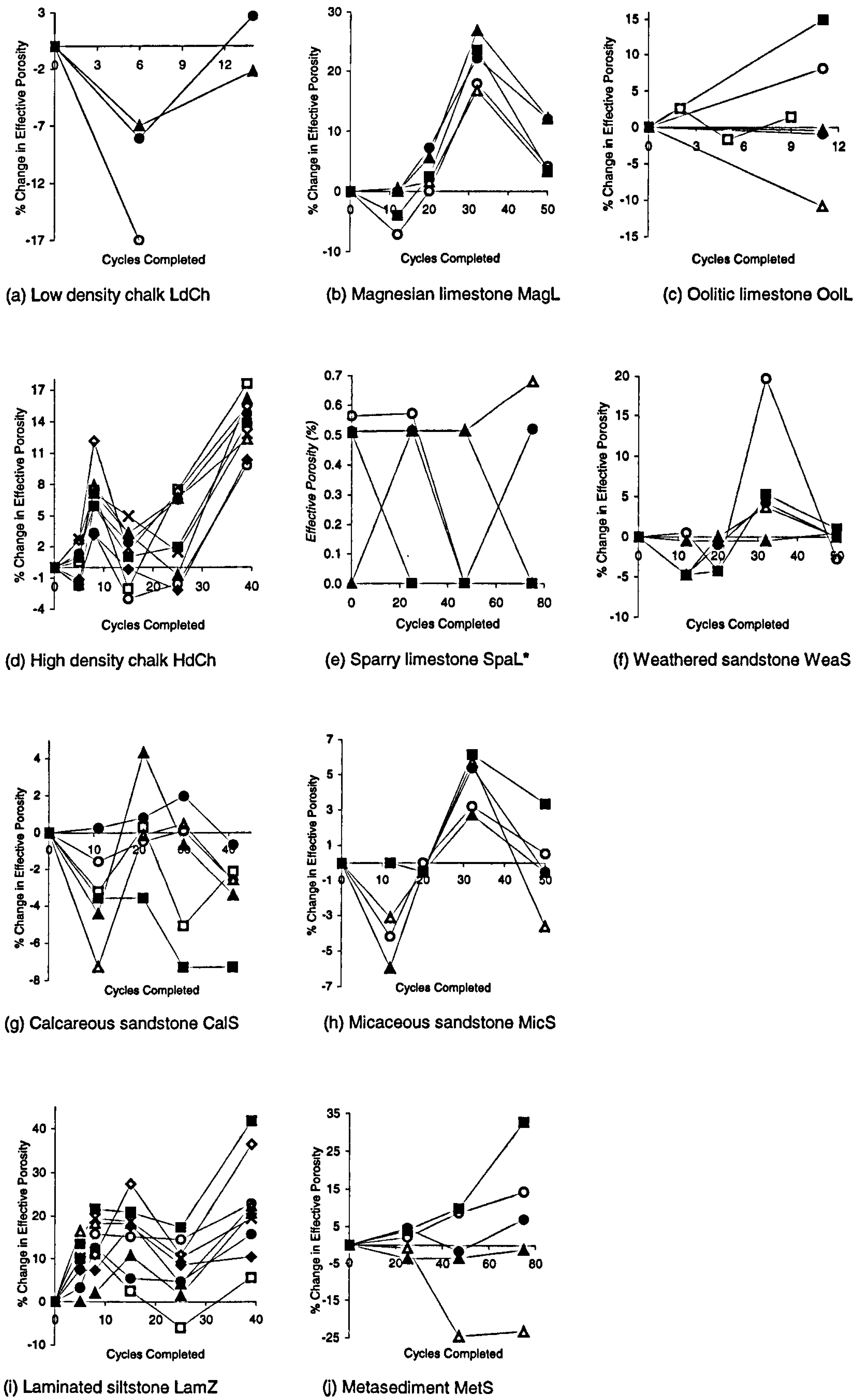


Figure 5.12 Percentage change in effective porosity due to freeze-thaw

**Absolute* effective porosity is plotted for SpaL because of the low values involved

Different symbols represent individual specimens

considerable temporal variation, notably MagL and MicS (Figure 5.12b and h). Discussion of the possible reasons for such variations is given in the context of modulus of elasticity measurements (section 5.5.6).

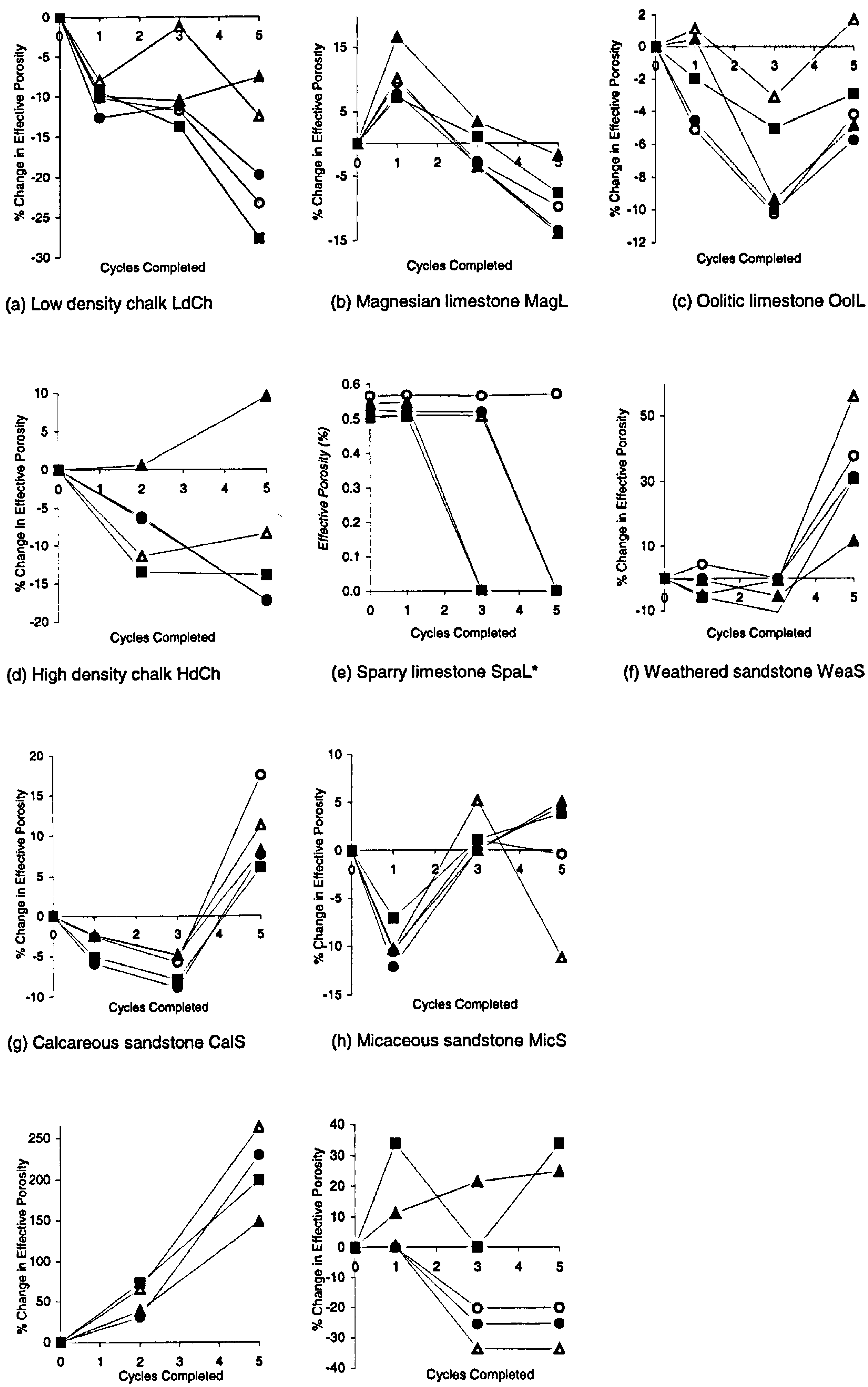
5.5.1.2 Salt weathering (Figure 5.13)

Rocks can be categorised into three groups in terms of their response to salt weathering. In the first group, WeaS, CalS and LamZ, n_e showed an absolute *increase* (Figures 5.13f, g and i) from 2% in CalS to 21% in LamZ. The first two of these experienced an initial reduction in n_e. Individual specimens for all three samples behaved in a consistent fashion. In the second group, LdCh, MagL, OolL, HdCh and SpaL, n_e showed an absolute *decrease* (Figures 5.13a, b, c, d and e) by around 0.5% in OolL and SpaL, 2-3% in MagL and HdCh, and 6% in LdCh. This is probably due to retention in pore spaces of crystallised salts and this would account for the weight gain measured for some of these rocks. Like the first group, there was close correspondence between individual specimens. Unusually, in MagL, all specimens showed an initial *increase* in n_e, followed by a subsequent reduction (Figure 5.13b). This might indicate that modification of the pore structure was necessary before salt solution was able to gain access to finer pores. In the third group, comprising MicS and MetS (Figures 5.13h and j) there was *no significant change* in n_e overall resulting from salt weathering. However, the trend of the mean sample value is slightly misleading in the case of MicS: It is notable that the temporal distribution of specimens for MicS (Figure 5.13h) is very similar to CalS (which showed a mean increase in n_e), but a single anomalous specimen has a significant effect on the mean value.

As noted above, some rocks which showed a post-test increase in n_e, actually showed the reverse trend in the early stages of the test (WeaS and CalS). The decreasing porosity trends for some rocks (eg OolL, HdCh and MicS) were also beginning to show a trend to increasing porosity by the latter stages of the test. This has several implications. (i) It can be inferred that had further cycles of salt weathering been conducted, some rocks which showed a post-test *decrease* in n_e, might have subsequently showed an increase. (ii) The rate at which modifications to porosity occurs differs in different rocks, with some clearly responding much more rapidly. (iii) By the end of testing, salt deposits might have been completely removed from WeaS since in addition to an increase in post-test n_e there was also a reciprocal reduction in ρ. In CalS, ρ showed a small increase suggesting that some salt deposits were retained. A fine coating of salt deposits on and between grains is evident from SEM analysis for several rocks, including CalS (Plate 5.20a, p.160), OolL (Plate 5.20b, p.160) and MicS (Plate 5.20c, p160).

5.5.1.3 Wetting and drying (Figure 5.14)

Of the samples tested for wetting and drying, two, HdCh and LamZ (Figure 5.14b and d), showed an increase in n_e while two others, LdCh and CalS (Figures 5.14a and c), showed a small reduction. While the former showed very strong consistency between specimens and little temporal variation, the latter were much more variable. The absolute increase in n_e for HdCh of 3.5% was bigger than the change for this sample under any other test. This is surprising since this rock did not indicate *any* macro changes under these test conditions (Figure 4.5 g, h and i).



(a) Low density chalk LdCh
(b) Magnesian limestone MagL
(c) Oolitic limestone OolL
(d) High density chalk HdCh
(e) Sparry limestone SpaL*
(f) Weathered sandstone WeaS
(g) Calcareous sandstone CalS
(h) Micaceous sandstone MicS
(i) Laminated siltstone LamZ
(j) Metasediment MetS

Figure 5.13 Percentage change in effective porosity due to salt weathering

**Absolute* effective porosity is plotted for SpaL because of the low values involved

Different symbols represent individual specimens

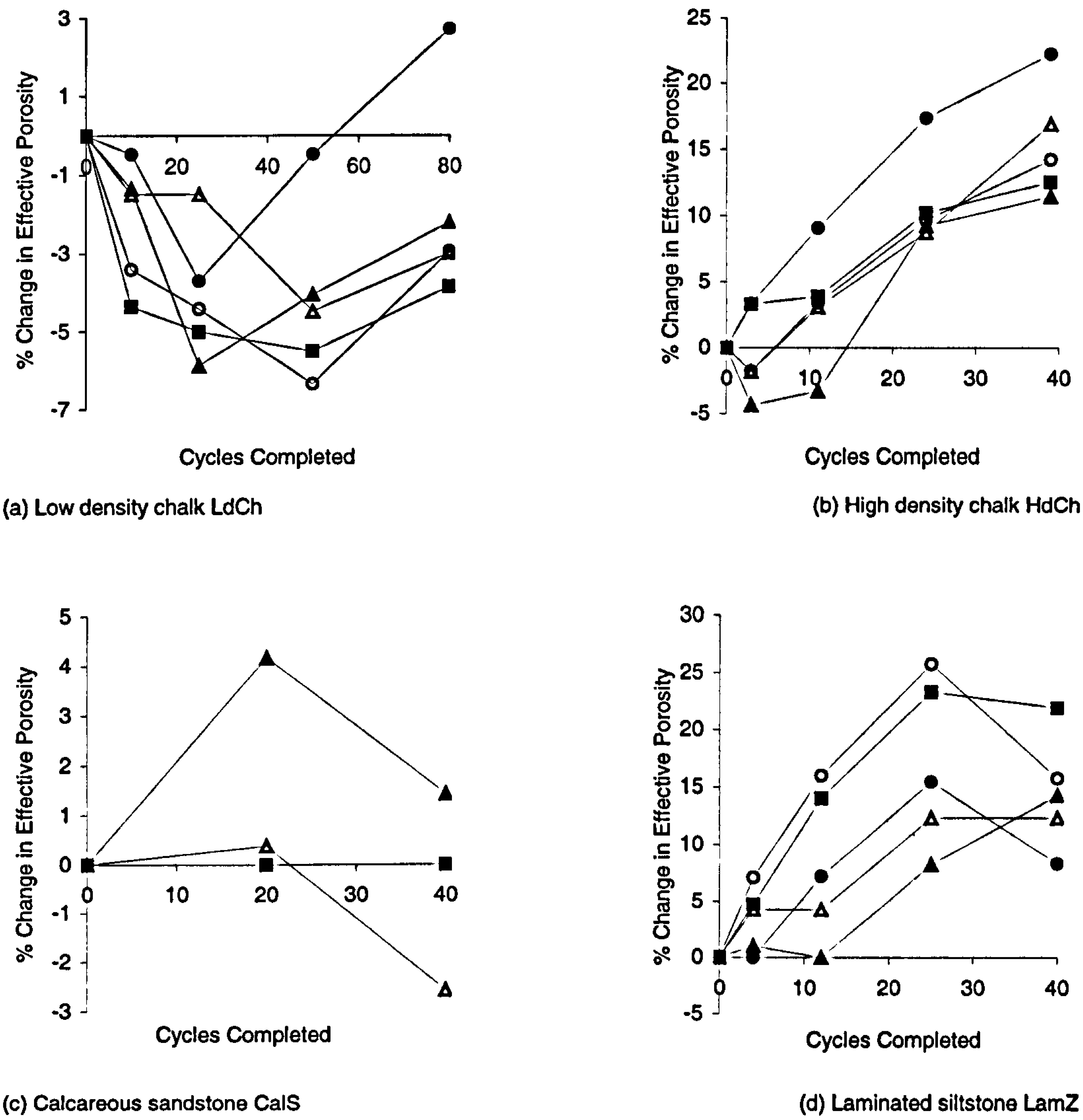


Figure 5.14 Percentage change in effective porosity due to wetting and drying

Different symbols represent individual specimens

5.5.1.4 Slake durability (Figure 5.15)

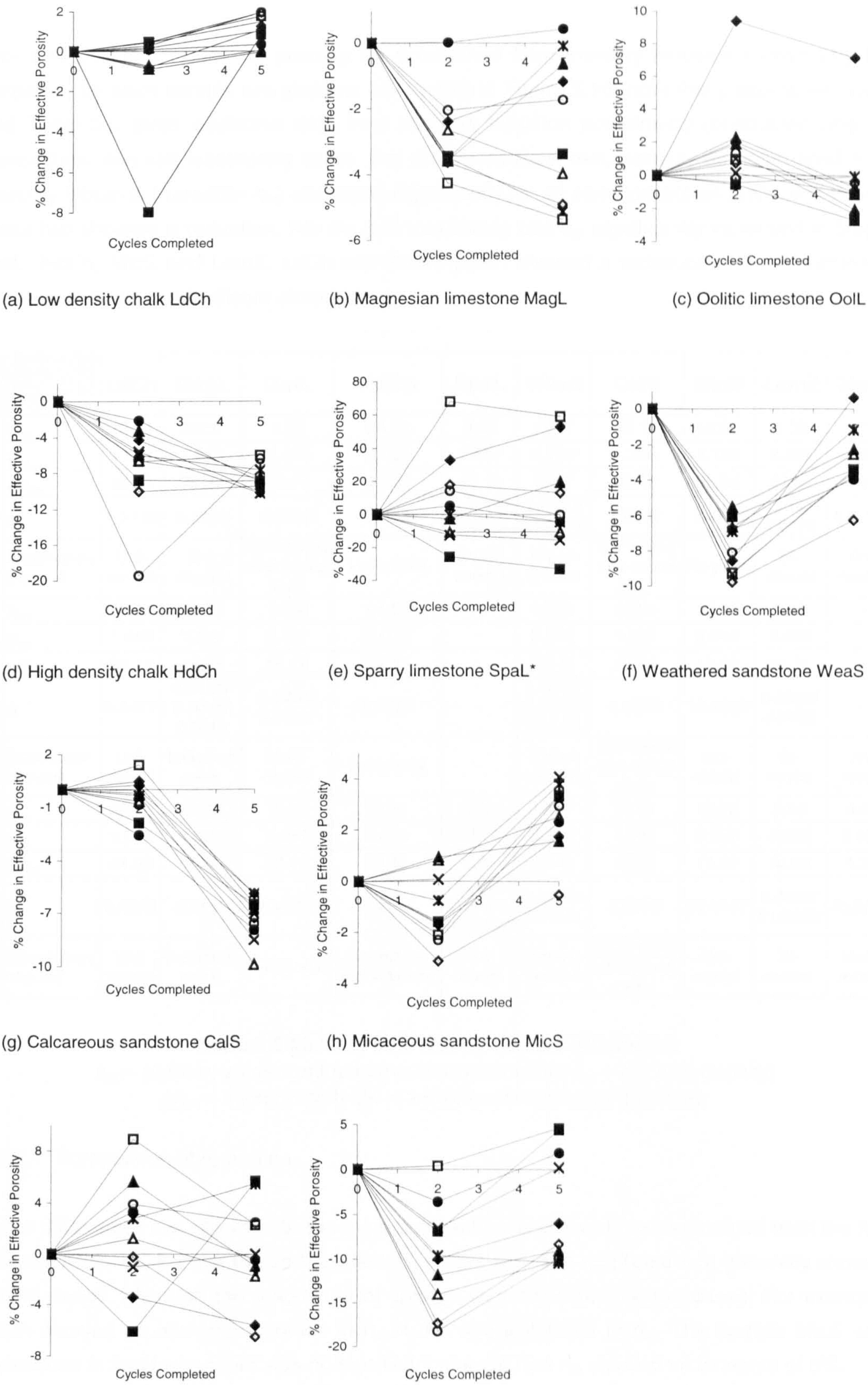
Following slake durability two samples showed a small reduction in n_e (HdCh, CalS) while the remaining samples showed no significant change. For both wetting and drying and slake durability tests there was no significant change in ρ for HdCh indicating that in both cases the change in n_e reflects changes in the degree of saturation achieved. For all samples there was close correspondence between individual specimens (Figure 5.15a to j).

Data*		LdCh	MagL	OolL	HdCh	SpaL	WeaS	CalS	MicS	LamZ	MetS	OolL(2)
Freeze-thaw												
Pre FT	n_e	34.17	14.57	17.22	21.96	0.42	10.83	18.23	15.56	6.45	1.06	18.14
	ρ	1.737	1.628	2.153	2.010	2.664	2.230	1.944	2.089	2.538	2.655	2.135
Post FT	n_e	33.59	15.59	17.55	24.94	0.24	10.80	17.67	15.52	7.87	1.13	18.40
	ρ	1.775	1.628	2.120	2.006	2.687	2.222	1.948	2.083	2.476	2.658	2.137
Salt weathering												
Pre MS	n_e	32.69	14.28	16.61	22.49	0.53	10.91	18.11	15.54	7.63	1.72	-
	ρ	1.749	1.614	2.172	2.008	2.661	2.226	1.968	2.096	2.512	2.640	-
Post MS	n_e	26.71	12.94	16.04	20.23	0.11	14.58	19.93	15.60	28.48	1.79	-
	ρ	1.900	1.634	2.224	2.117	2.681	2.132	2.013	2.092	2.092	2.630	-
Wetting and drying												
Pre WD	n_e	32.85	-	-	21.72	-	-	18.13	-	6.91	-	-
	ρ	1.742	-	-	2.002	-	-	1.947	-	2.506	-	-
Post WD	n_e	32.22	-	-	25.04	-	-	17.55	-	7.92	-	-
	ρ	1.738	-	-	2.012	-	-	1.953	-	2.477	-	-
Slake durability												
Pre SD	n_e	33.83	18.36	17.40	23.55	0.65	11.80	20.48	16.67	8.17	2.21	-
	ρ	1.657	1.662	2.203	2.019	2.687	2.252	1.947	2.021	2.532	2.645	-
Post SD	n_e	34.18	17.90	17.37	21.55	0.67	11.44	18.99	17.09	8.17	2.10	-
	ρ	1.663	1.650	2.197	2.020	2.693	2.240	1.944	2.021	2.508	2.651	-

Table 5.2 Pre and post-test effective porosity and dry bulk density
* FT = Freeze-thaw; MS = salt weathering (magnesium sulphate);
WD = wetting and drying; SD = slake durability

5.5.1.5 The role of rock properties in n_e modification

In some rocks (eg MicS, MetS) pore modifications did not occur for any test despite evidence that deterioration occurred. Other rocks seemed pre-disposed to pore modification regardless of the weathering test (eg HdCh, CalS, LamZ). These results suggest that pore modification is more closely related to rock material properties than to the weathering processes involved. There appears to be some correlation, for instance, between pore modification and pore size distribution (Table 3.4). Rocks with a small modal pore diameter and high μn_m, for instance, are also those where significant changes in n_e occurred after freeze-thaw (LdCh, MagL, HdCh and LamZ). Equally, rocks in which pore infilling after salt weathering could be inferred by a reduction in n_e and an increase in ρ, also had a high proportion of fine pores (LdCh, MagL, HdCh). Notably, pore infilling is not inferred from reductions in n_e for any of the granular rocks with coarse pores. This might indicate the importance of capillarity in drawing fluid into the finest pores and points indirectly to the importance of other rock properties such as μn_m, pore size distribution and saturation coefficient.



(a) Low density chalk LdCh
(b) Magnesian limestone MagL
(c) Oolitic limestone OolL
(d) High density chalk HdCh
(e) Sparry limestone SpaL*
(f) Weathered sandstone WeaS
(g) Calcareous sandstone CalS
(h) Micaceous sandstone MicS
(i) Laminated siltstone LamZ
(j) Metasediment MetS

Figure 5.15 Percentage change in effective porosity due to slake durability

Different symbols represent individual specimens

5.5.2 Porosity determined from mercury intrusion (n_m)

Pre- and post-test values for porosity as determined from mercury intrusion porosimetry (n_m hereafter) for each sample are given as histograms in Figure 5.16 (note that y axis scales vary) and Table 5.3 gives additional data from mercury intrusion porosimetry (conducted only for freeze-thaw and salt weathering tests). For the freeze-thaw test, porosity as determined from mercury intrusion (hereafter n_m) increased significantly for all samples except LdCh and WeaS, these two showing a reduction. For the salt weathering test n_m significantly increased in MagL, OolL, HdCh, MicS and LamZ. LdCh and WeaS again showed a reduction and the remaining samples showed no significant change.

MIP		LdCh	MagL	OolL	HdCh	SpaL	WeaS	CalS	MicS	LamZ	MetS
Pre-Test	n_m	38.58	33.61	7.62	24.23	0.60	14.17	26.15	12.53	5.39	0.37
	ρ_m	1.610	1.819	2.478	2.007	2.684	2.236	1.828	2.195	2.599	2.695
	μn_m	65.58	75.27	93.18	99.59	18.18	19.87	29.42	35.38	97.58	0.00
	ϕ	1.0143	0.3533	0.5702	0.4058	0.1076/ 3.5145	1.7829/ 21.8436	2.9947	7.4786	0.0141/ 0.0175	43.2507
	Description of p.s.d	Uni-modal	Uni-modal	Uni-modal with broad peak	Uni-modal	Multi-modal	Broad spread	Uni-modal with broad peak	No peak	Bi-modal	Uni-modal
Freeze Thaw	n_m	32.31	40.71	18.34	24.53	-	12.95	26.90	18.73	5.55	-
	ρ_m	1.844	1.584	2.187	2.029	-	2.250	1.927	2.049	2.669	-
	μn_m	93.15	49.88	68.18	99.83	-	20.49	47.71	17.18	86.36	-
	ϕ	0.8477	0.6183/ 0.7015/ 2.9815	0.5708/ 2.7515	0.4881	-	1.7814/ 8.7838/ 10.184	1.5073	10.8022	0.0349/ 0.0462	-
	Description of p.s.d	Uni-modal	Indistinct peak	Multi-modal	Uni-modal	-	Broad spread	Uni-modal with broad peak	Uni-modal	Bi-modal	Uni-modal
Salt Weathering	n_m	22.15	42.08	15.47	25.31	0.32	13.07	25.61	19.12	8.66	0.61
	ρ_m	2.059	1.579	2.251	2.003	2.679	2.219	1.868	2.087	2.508	2.683
	μn_m	94.98	69.03	93.45	99.45	0.00	24.62	45.59	14.63	66.38	4.35
	ϕ	0.4578	0.6181	0.4575	0.4146	14.4061/ 21.6184/ 43.2507	1.2907/ 8.7838	2.8575	14.4107	0.0346/ 21.6288	43.2507
	Description of p.s.d	Uni-modal	Indistinct peak	Uni-modal	Uni-modal and subdued	Uni-modal	Broad spread	Uni-modal with broad peak	Uni-modal	Bi-modal	Multi-modal

Table 5.3 Mercury porosimetry data for all samples
n_m = porosity as determined by mercury intrusion; ρ_m = dry bulk density;
μn_m = microporosity (% < 1μm); ϕ = modal pore diameter.

5.5.2.1 Comparison of n_e and n_m

Some differences are apparent between porosity values and trends as determined from the two different methods. For the freeze-thaw test, n_e (Table 5.2) and n_m (Table 5.3) generally showed similar trends, although the magnitude of change was different in some cases. For example, HdCh showed an absolute increase in n_e of 3% and just 0.3% in n_m. The sample MicS was anomalous in that for n_e there was no significant change, but n_m showed an increase of 6%.

For the salt weathering test half of the samples showed similar trends (LdCh, SpaL, MicS, LamZ, MetS) though the degree of magnitude differed for MicS and LamZ. For the remaining rock types (MagL, OolL, HdCh, WeaS, CalS) there was a reverse trend between n_e and n_m.

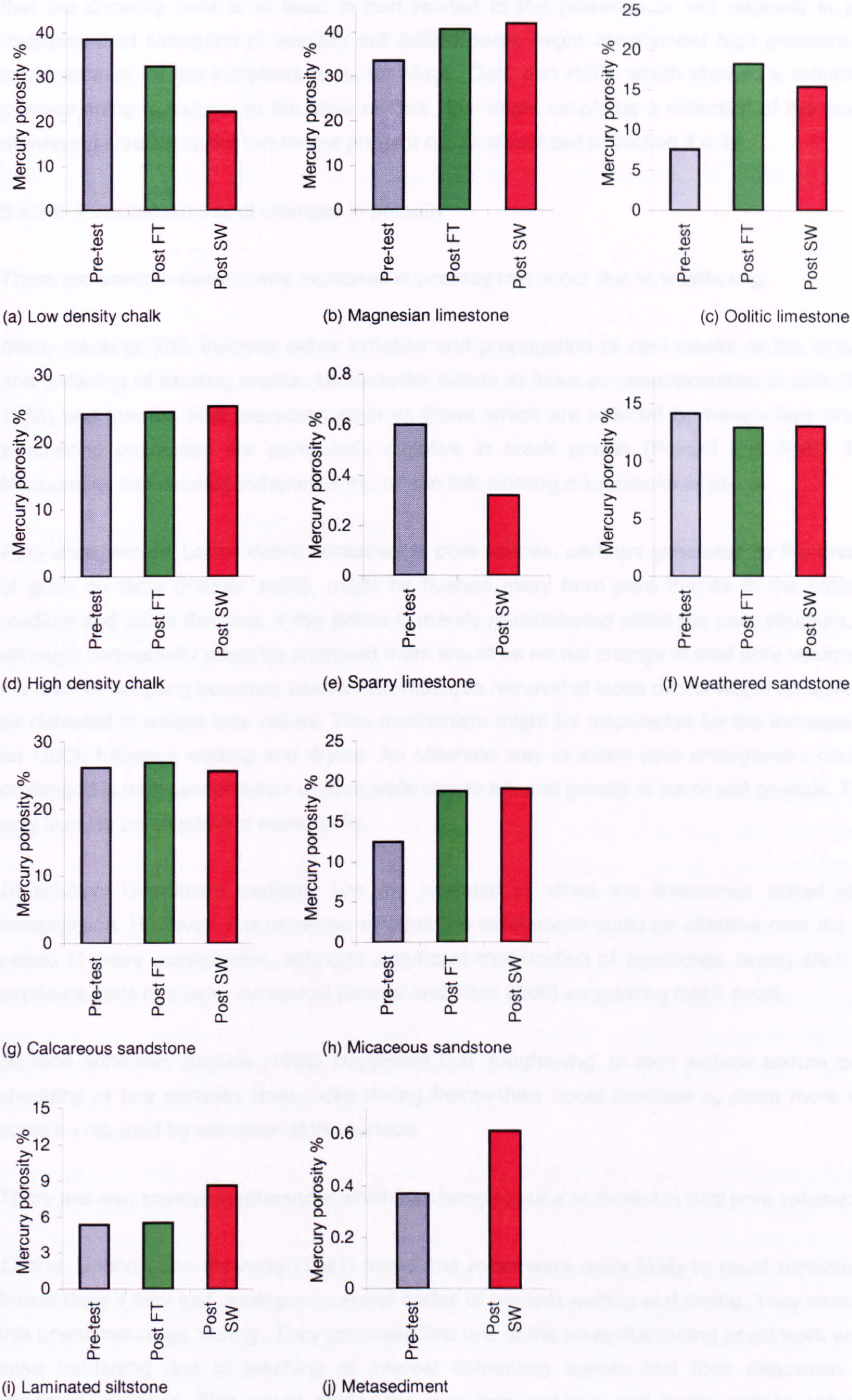


Figure 5.16 Histograms of pre and post-test mercury porosity (n_m)
FT = Freeze-thaw; SW = salt weathering

Given the much better correspondence between n_e and n_m for the freeze-thaw test it is probable that the disparity here is at least in part related to the presence of salt deposits in pores. Improved fluid saturation of partially salt infilled pores might occur under high pressure. This could account for the increases in n_m for MagL, OolL and HdCh which showed a reduction in corresponding n_e values. In the case of OolL, this might simply be a reflection of the use of a non-representative specimen for the pre-test n_m, as discussed in section 4.4.4.

5.5.2.2 Potential causes of changes in porosity

There are several reasons why *increases* in porosity can occur due to weathering:

Micro-cracking: This involves either initiation and propagation of new cracks or the extension and widening of existing cracks. Microcracks initiate at flaws or heterogeneities in rock (Kranz 1983) and internal fluid pressures such as those which are induced by freeze-thaw and salt weathering processes are particularly effective in crack growth (Pollard and Aydin 1988). Microcracks can develop independently, or can link existing microcracks or pores.

Pore enlargement: Loose debris contained in pore spaces, perhaps generated by the break up of grain contacts (Fitzner 1988), might be flushed away from pore throats in the saturating medium and out of the rock. If this debris is merely re-distributed within the pore structure, then although connectivity *could* be improved there would be no net change in total pore volume. For the level of weighing accuracy used in this research removal of loose debris would be unlikely to be detected in weight loss values. This mechanism might be responsible for the increase in n_e for HdCh following wetting and drying. An alternate way in which pore enlargement could be envisaged is from compression of pore walls due to internal growth of ice or salt crystals. This is only likely to be effective in weak rocks.

Dissolution: Dissolution certainly has the potential to affect the limestones tested in this investigation. However, it is uncertain whether the mechanism could be effective over the short period of these experiments, although significant modification of limestones during short term exposure trials has been measured (Moses and Viles 2000) suggesting that it could.

Surface adhesion: Brockie (1972) suggested that 'roughening' of rock surface texture due to shedding of fine particles from rocks during freeze-thaw could increase n_e since more water could be retained by adhesion at the surface.

There are also several mechanisms which can bring about a *reduction* in total pore volume:

Curing: Lienhart and Stransky (1981) found that rocks were more likely to resist experimental freeze-thaw if they had undergone several cycles of pre-test wetting and drying. They described this phenomenon as 'curing'. They proposed that one of the ways that curing could work was by case hardening due to leaching of internal cementing agents and their deposition onto specimen surfaces. This would render the rock less pervious and hence reduce saturation efficiency, although total pore volume might be unaffected. This mechanism could explain the reduction in n_e for HdCh following slaking.

Deposition of crystallised salts in pores: The uptake of salts and the binding effect which sometimes results are well known phenomena (eg Goudie 1974; Booth 1990; McGreevy 1996; Williams and Robinson 1998). Pore infilling appears to occur for many of the rocks tested here and explains why some of these showed weight gains.

Pore compression: Collapse or compression of pores might also occur, particularly in weaker rocks (eg LdCh and MagL). This could occur, for example, by external ice pressure from the surface.

5.5.3 Pore size distribution and microporosity (μn_m)

Pre and post-test pore size distributions for the freeze-thaw and salt weathering tests are given in Figures 5.17 to 5.26. The y axis 'incremental porosity %' used on these charts refers to the actual rock porosity (mercury derived) corresponding to each pore throat diameter class and is found from:

$$\text{Increm. porosity}\,\% = \frac{\text{Increm. volume (mL/g)}}{\text{Total intrusion volume (mL/g)}} * n_m\ \% \qquad [5.1]$$

Pore size distribution obtained from mercury porosimetry show that each sample responded uniquely to freeze-thaw and salt weathering.

5.5.3.1 Freeze-thaw

MagL, OolL and MicS showed an increase in the proportion of coarser pores, particularly in the range 1-10μm. There was a significant post-test change in the peak pore size and general shape of the distribution for MagL. This suggests that modification of existing pores occurred. If the increase in n_m, reported earlier, could be accounted for solely by new void space the pre-test peak in the distribution would still be apparent in the post-test distribution. The fact that this is not the case indicates that modification of existing pores has taken place. This might have been achieved by pore enlargement due to pore coalescence or the flushing out of loose debris. Had any loose debris simply been re-distributed, an increase in the proportion of finer pores could have been expected. There is some evidence of pore coalescence and linking of microcracks in SEM micrographs (Plate 5.20d, p160).

In the case of OolL and MicS there is evidence that much of the increase in n_m was achieved by the introduction of new void space. In OolL, this is indicated by the increased contribution to absolute porosity of pores of all sizes. It is not possible to say on the basis of the pore size distribution alone, to what extent existing pores were modified in OolL, but the larger increase in coarser pores can probably be interpreted in the same way as for MagL. In MicS too, the pre-test distribution remains essentially unchanged except for the addition of a new and dominant peak diameter at the coarse end of the range.

Since there is a small reduction in ρ for MicS it is reasonable to assume that the large increase in n_m is attributable, at least in part, to the introduction of new void space. Data from n_e and ρ

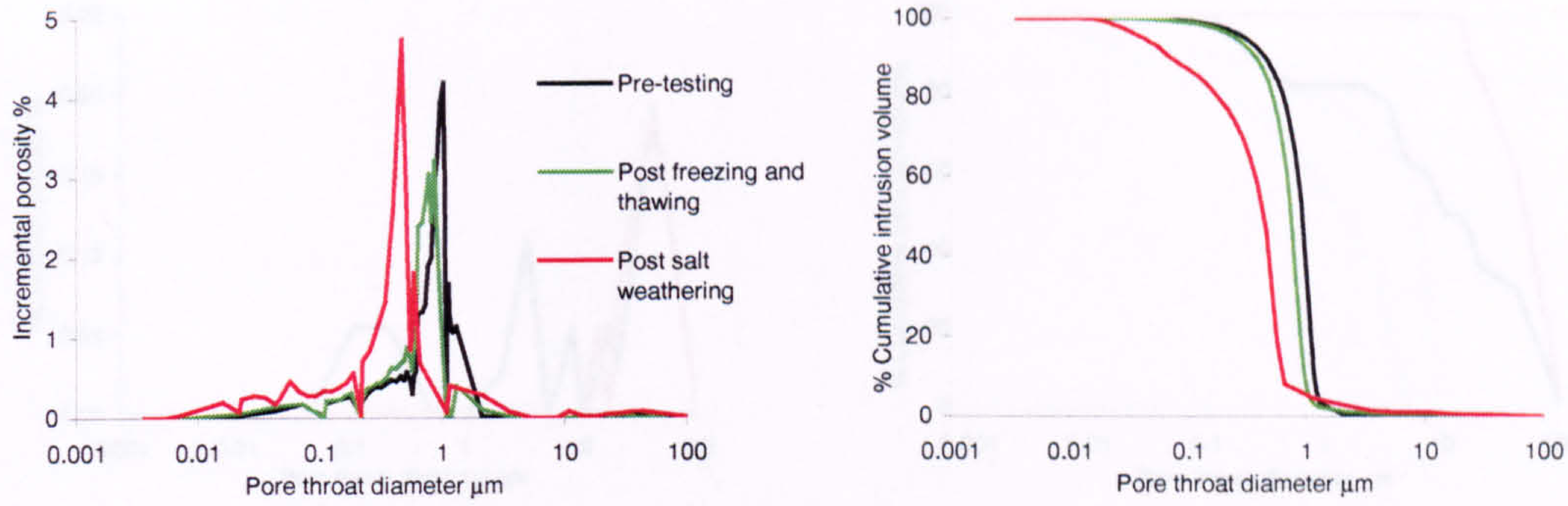


Figure 5.17 Pore size distribution for the low density chalk LdCh

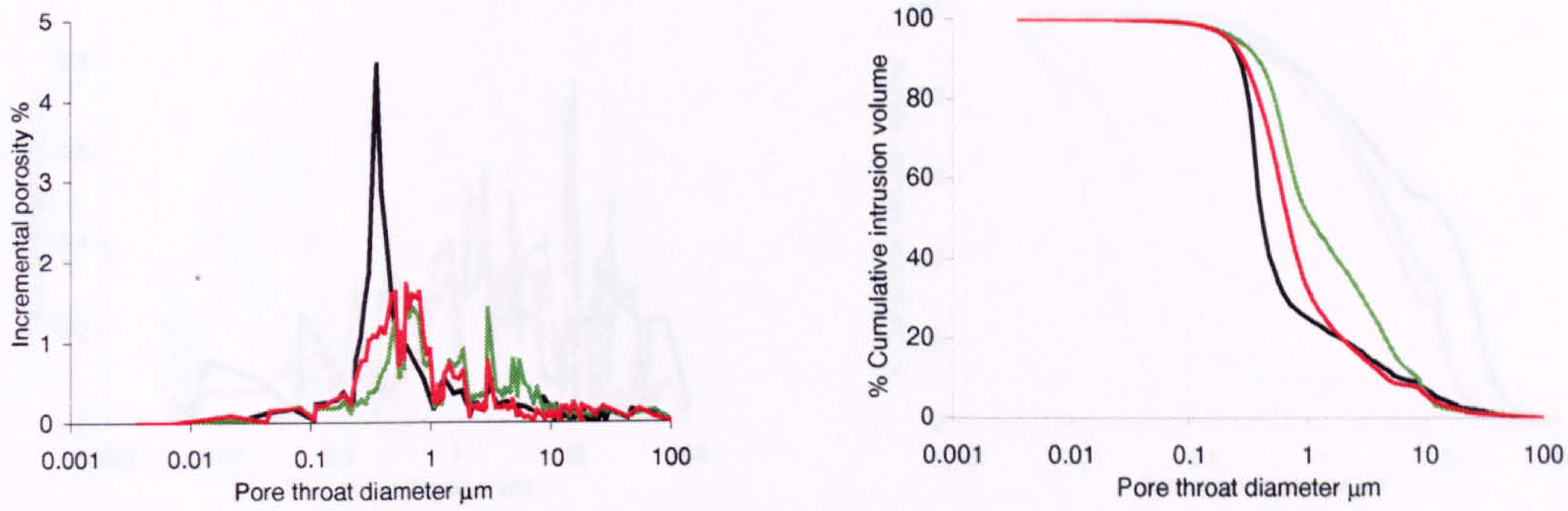


Figure 5.18 Pore size distribution for the magnesian limestone MagL

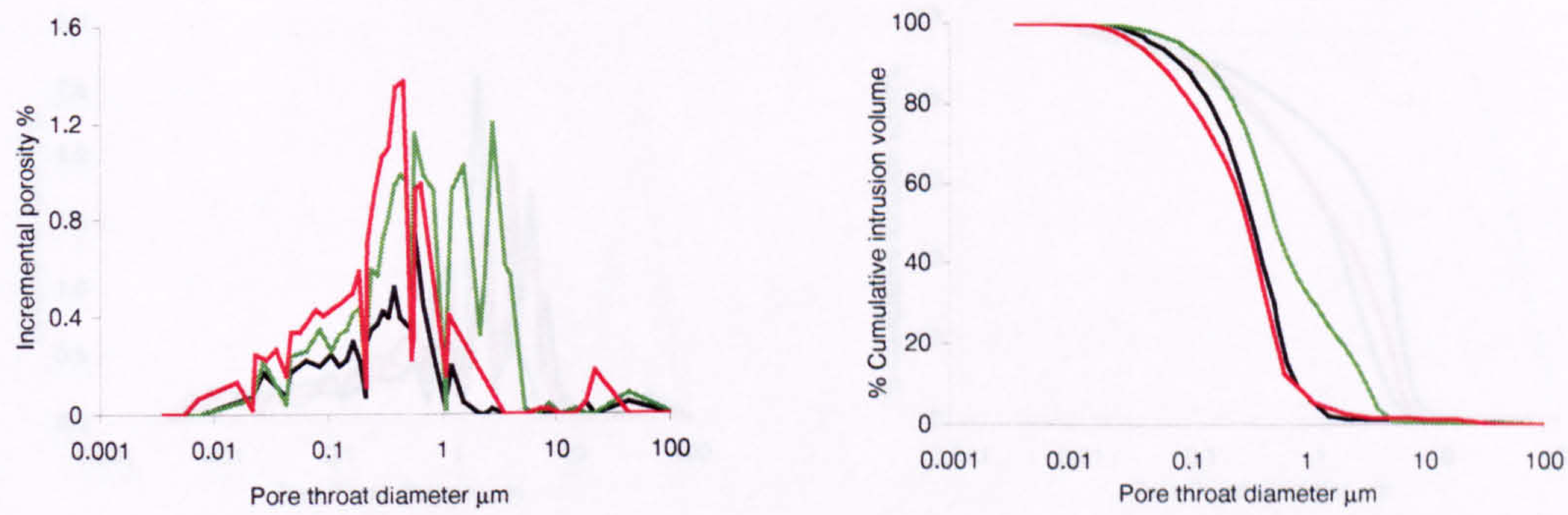


Figure 5.19 Pore size distribution for the oolitic limeston OolL

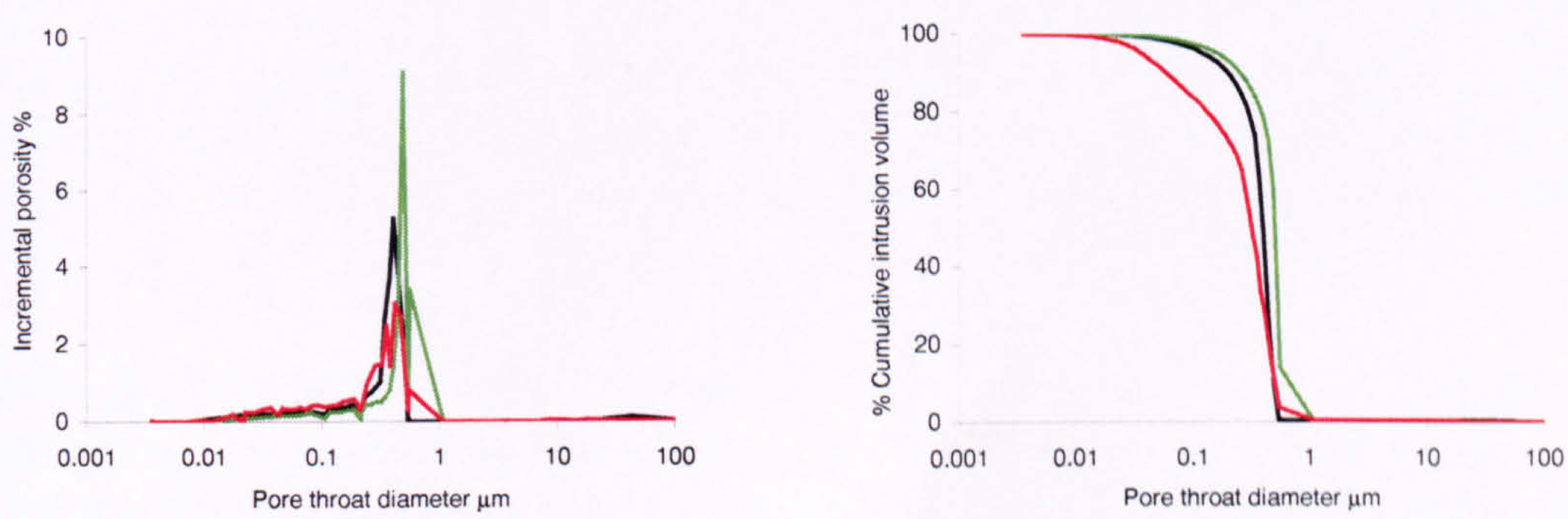


Figure 5.20 Pore size distribution for the high density chalk HdCh

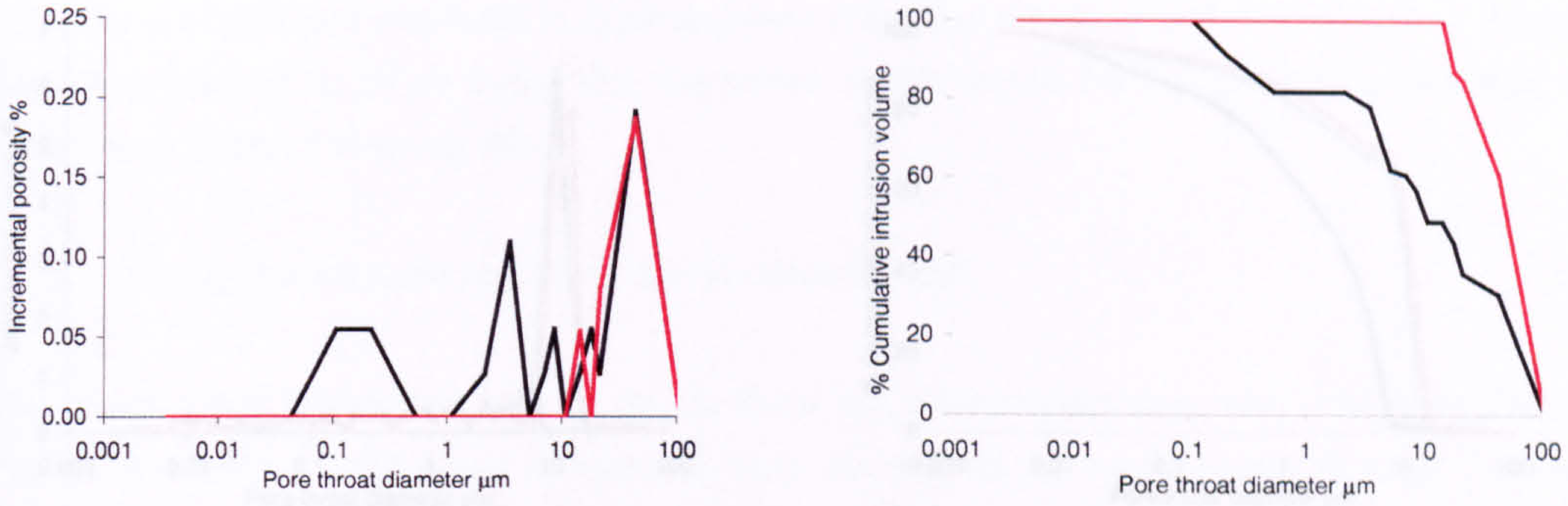


Figure 5.21 Pore size distribution for the sparry limestone SpaL

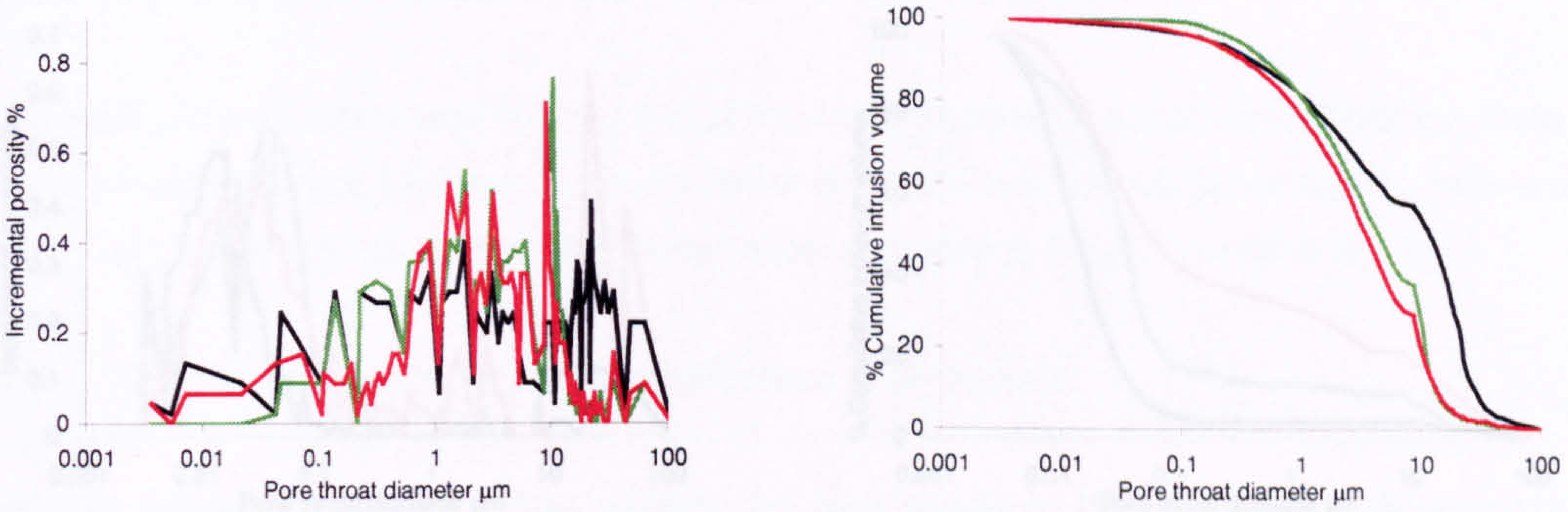


Figure 5.22 Pore size distribution for the weathered sandstone WeaS

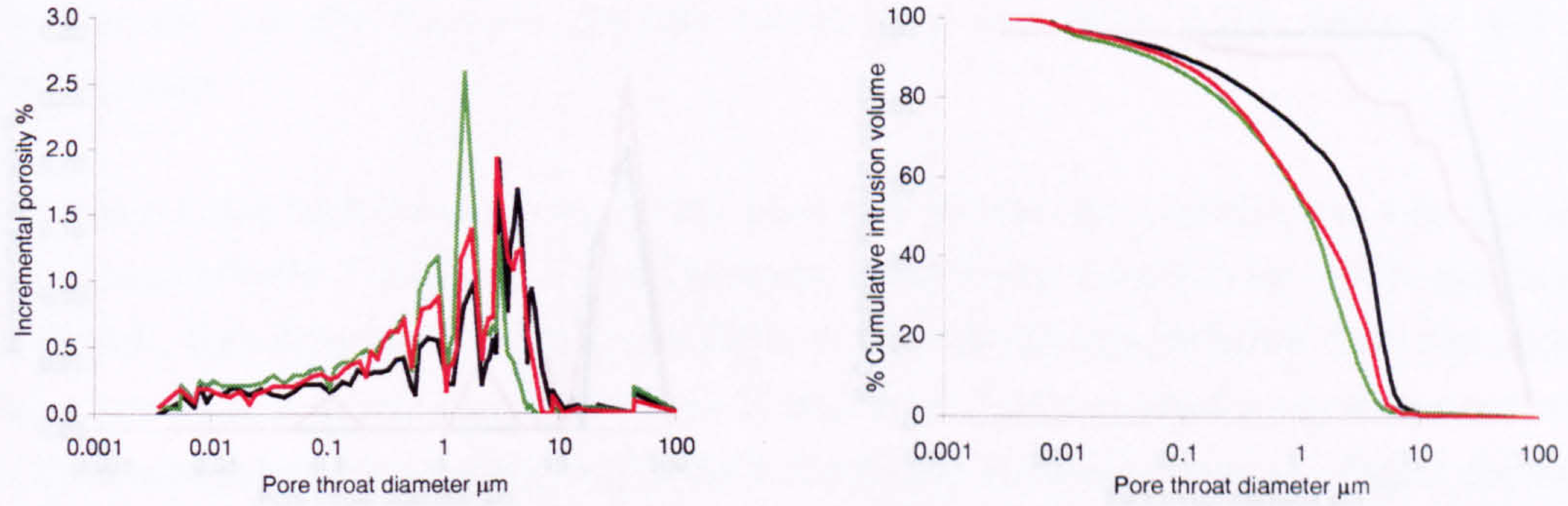


Figure 5.23 Pore size distribution for the calcareous sandstone CalS

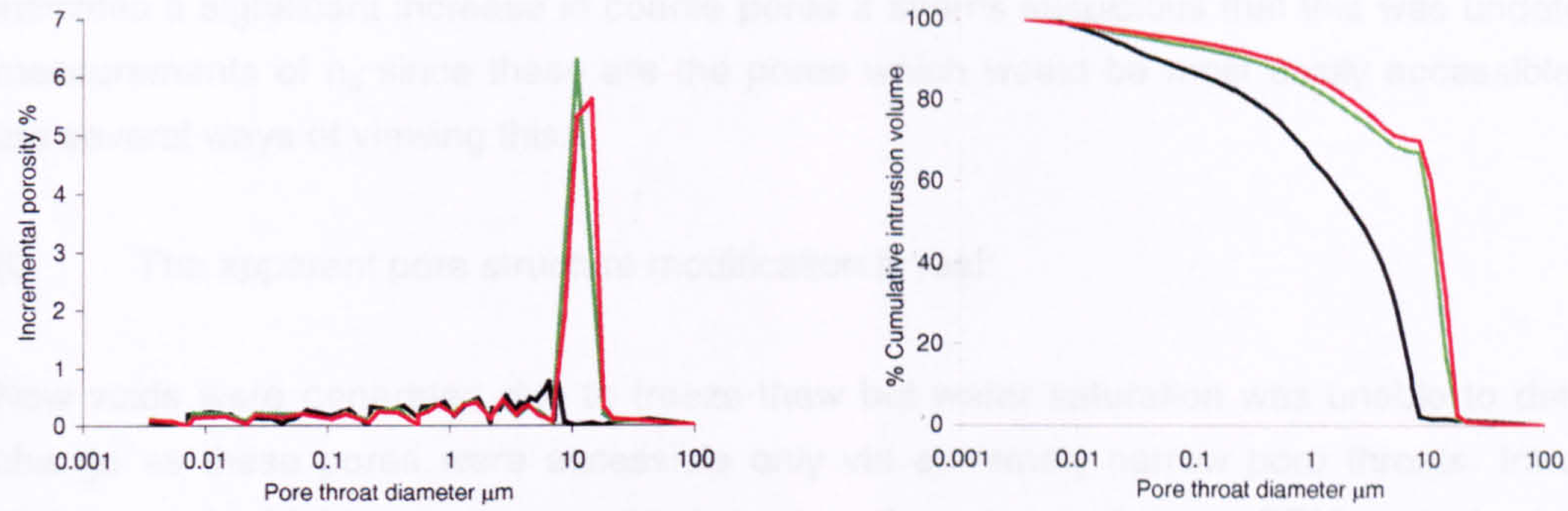


Figure 5.24 Pore size distribution for the micaceous sandstone MicS

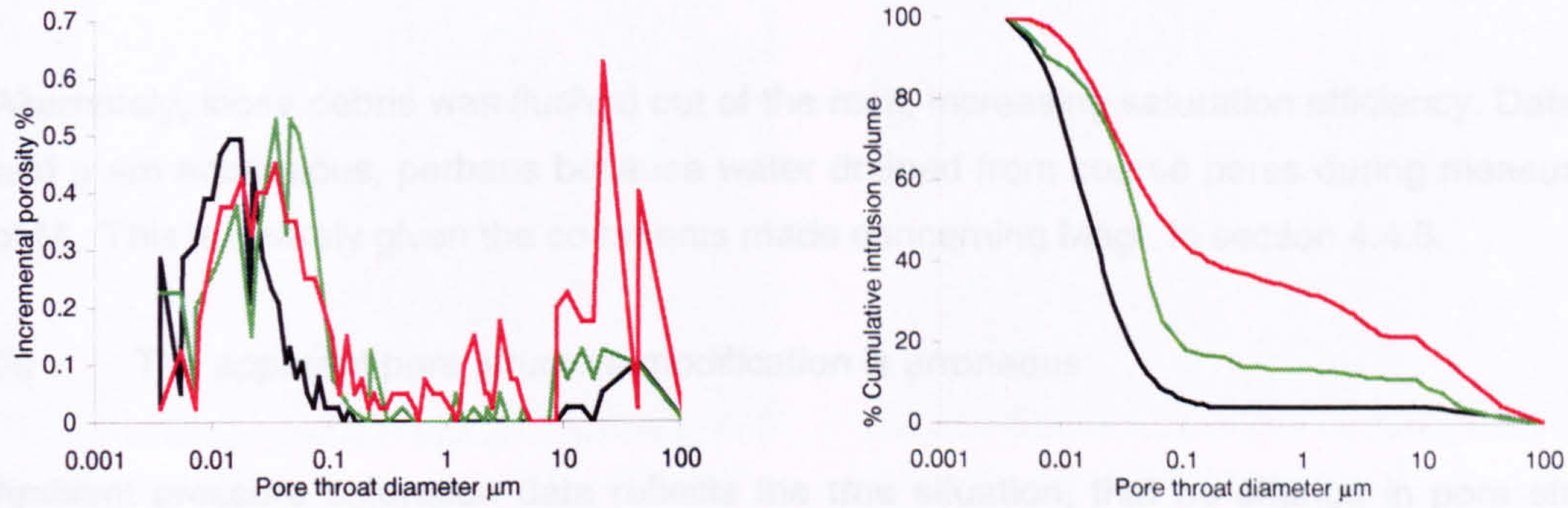


Figure 5.25 Pore size distribution for the laminated siltstone LamZ

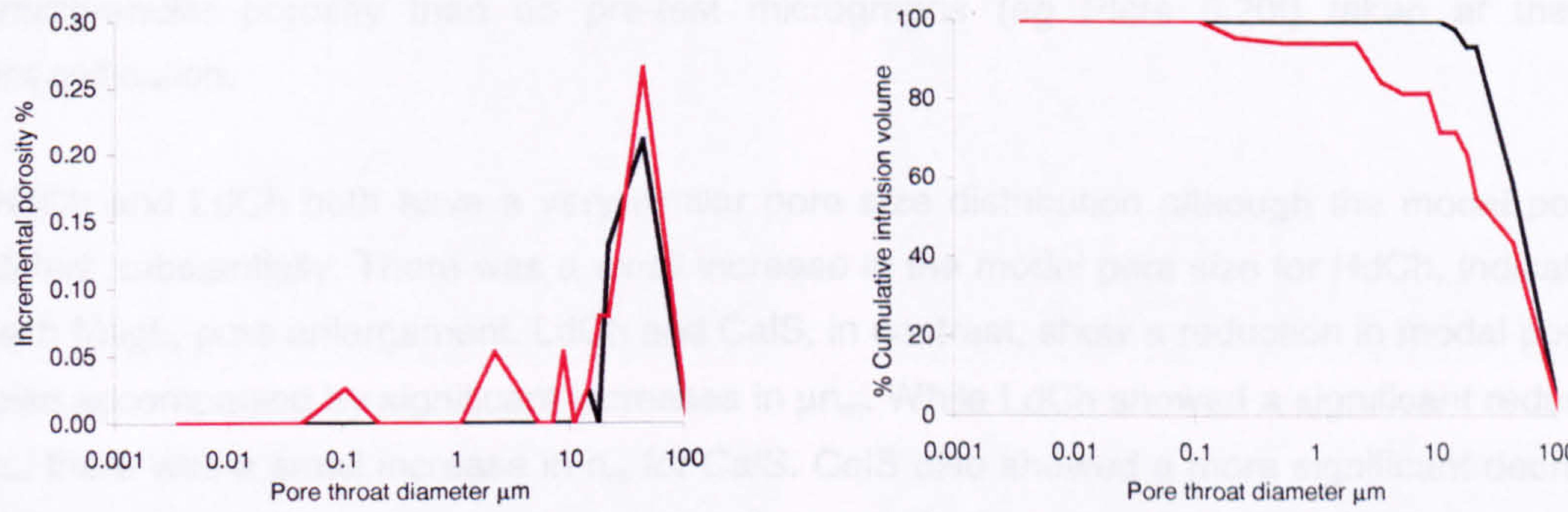


Figure 5.26 Pore size distribution for the metasediment MetS

however, indicate no change to the pore structure at all. Given that the pore size distribution indicates a significant increase in coarse pores it seems suspicious that this was undetected in measurements of n_e since these are the pores which would be most easily accessible. There are several ways of viewing this:

(i) The apparent pore structure modification is real:

New voids were generated due to freeze-thaw but water saturation was unable to detect any change as these pores were accessible only via extremely narrow pore throats. Intrusion of mercury under high pressure was able to access these trapped pores. SEM analysis shows that many new, apparently isolated microcracks were formed in MicS, lending some support to this theory (Plate 4.3g). The positive value for fracture porosity reported in Table 4.1 also lends support to the idea that a real increase in void volume occurred.

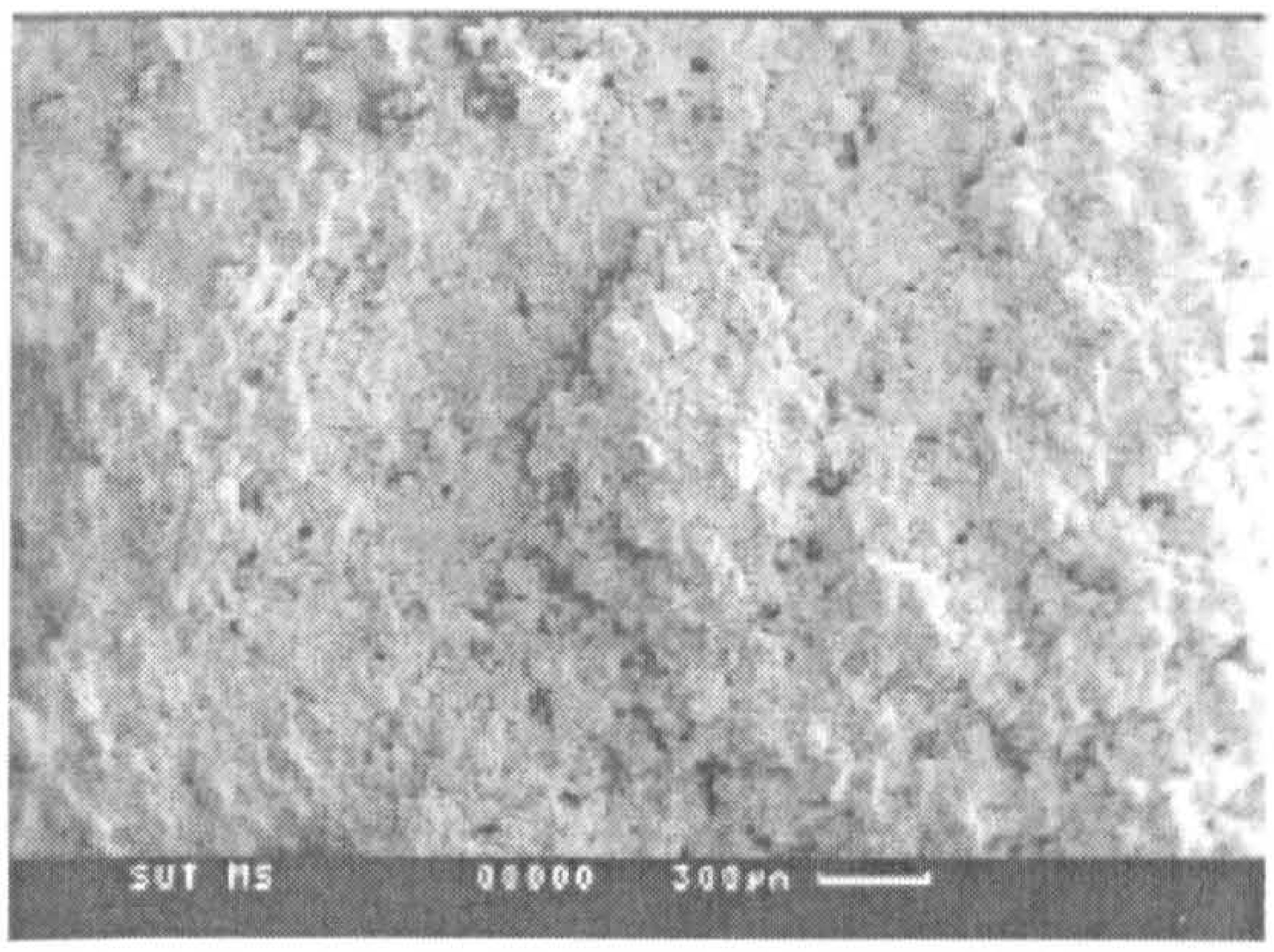
Alternately, loose debris was flushed out of the rock, increasing saturation efficiency. Data for n_e and ρ are anomalous, perhaps because water drained from coarse pores during measurement of M_s. This is unlikely given the comments made concerning MagL in section 4.4.5.

(ii) The apparent pore structure modification is erroneous:

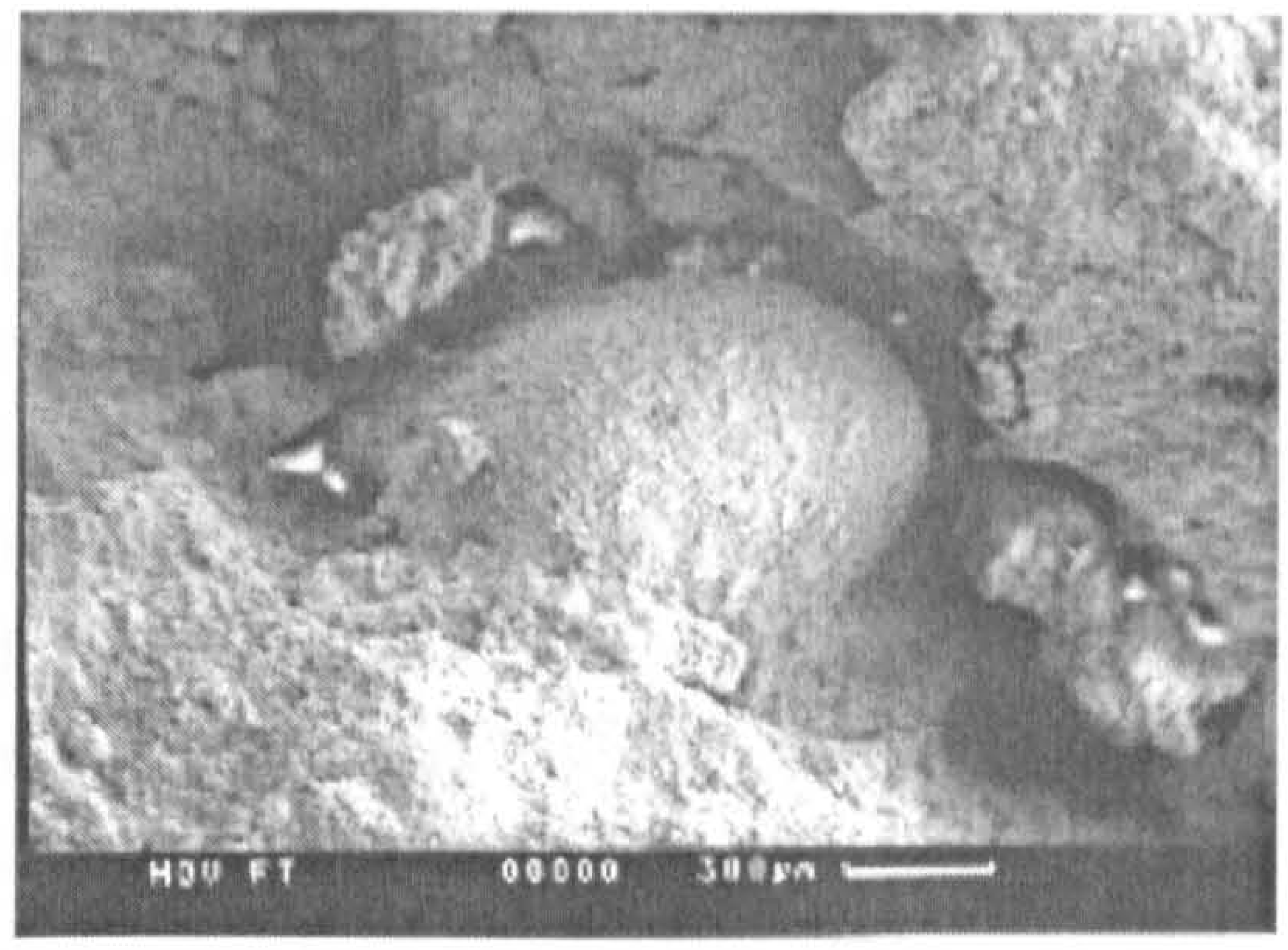
Ambient pressure saturation data reflects the true situation, that no change in pore structure occurred, and the pre-test pore size distribution is anomalous, perhaps due to use of a non-representative specimen. The low pre-test value for n_m compared to n_e and vacuum saturation data lend some support to this. Post-test SEM micrographs (eg Plate 5.20e) indicate a higher intergranular porosity than do pre-test micrographs (eg Plate 5.20f) taken at the same magnification.

HdCh and LdCh both have a very similar pore size distribution although the modal pore size differs substantially. There was a small increase in the modal pore size for HdCh, indicating, as with MagL, pore enlargement. LdCh and CalS, in contrast, show a reduction in modal pore size, also accompanied by significant increases in μn_m. While LdCh showed a significant reduction in n_m there was a small increase in n_m for CalS. CalS also showed a more significant decrease in the proportion of finer pores as well as an increase in the general spread of finer pores. These distributions suggest that either pore throat blockage has occurred, perhaps by debris re-distribution, or that pores have been infilled or even collapsed or compacted in some way. There is some suggestion of pore deformation in CalS in the SEM micrograph shown in Plate 5.20g. Case hardening of the type described by Lienhart and Stransky (1981) could be responsible for the reduction in n_m shown in LdCh but would not explain the increase in finer pores. For both these rocks, though, there would appear to be no net loss or gain of material or pore volume.

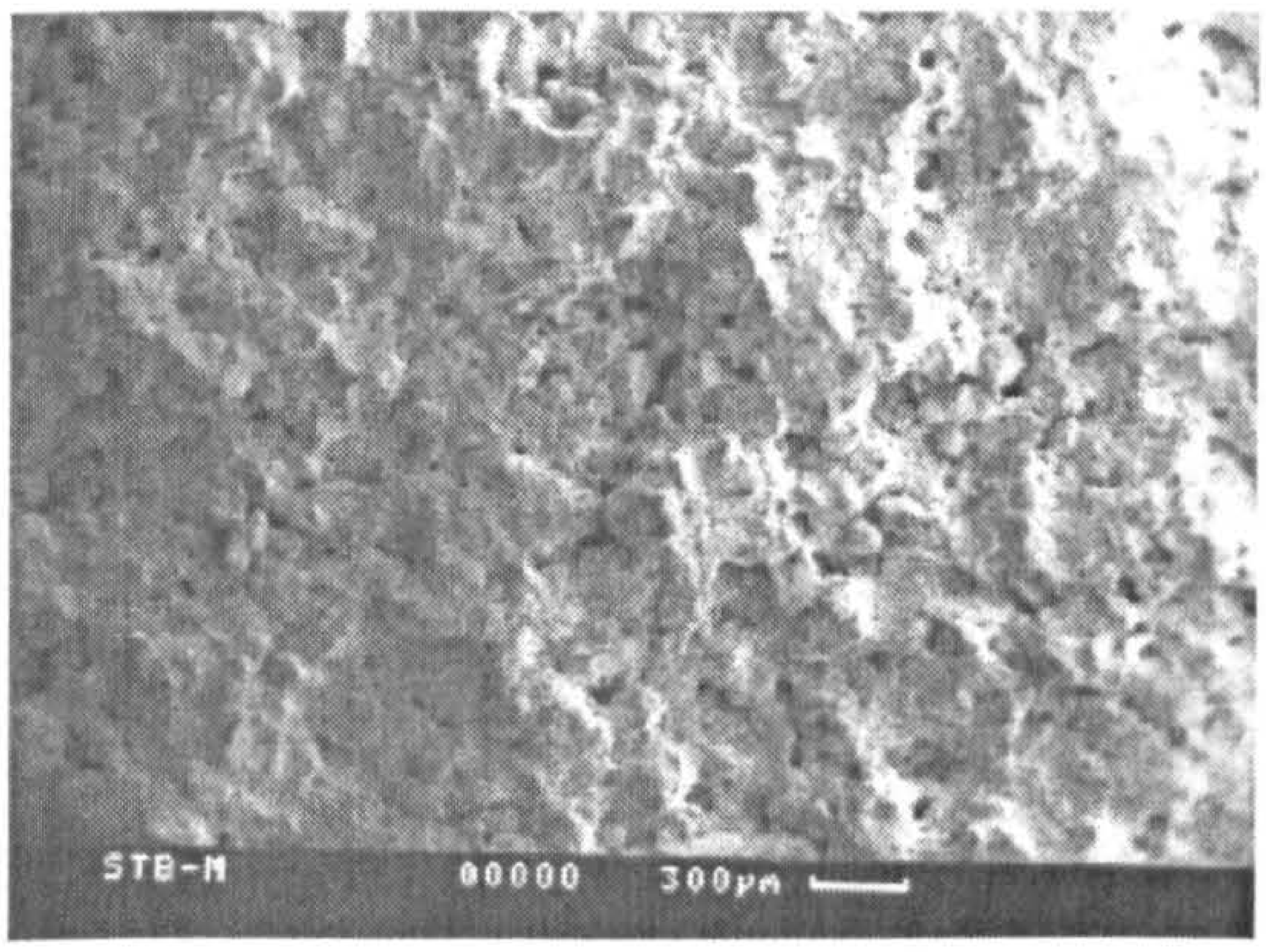
Although ostensibly, pore size distributions of WeaS and LamZ are very different, their responses to freeze-thaw have much in common. In both cases, there was reduction in both fine and coarse pores, and an increase in pores in the middle of the range. The reduction in fine pores suggests that coalescence or other enlargement of existing pores has occurred.



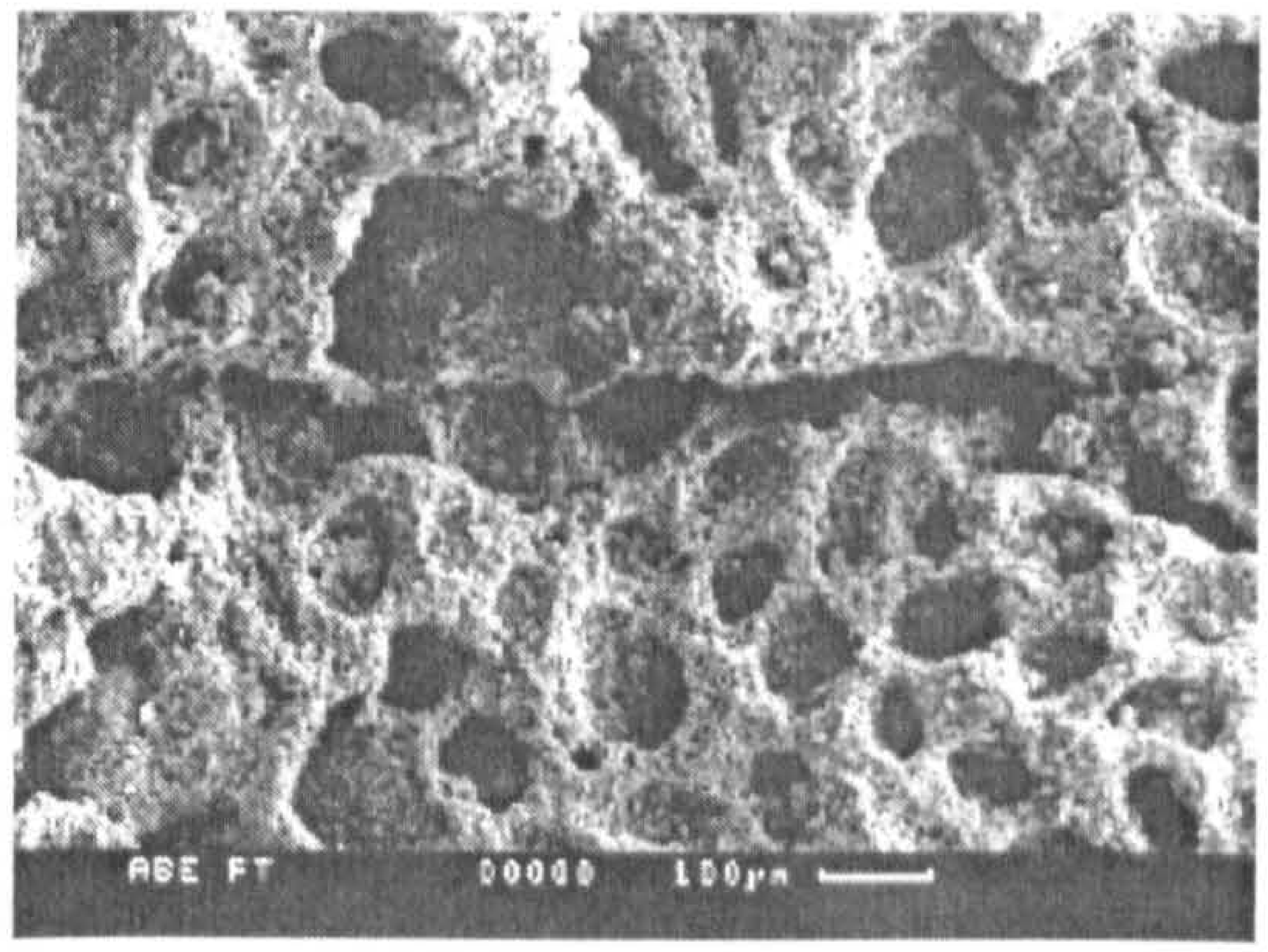
(a) Calcareous sandstone: Post SW (300μm)



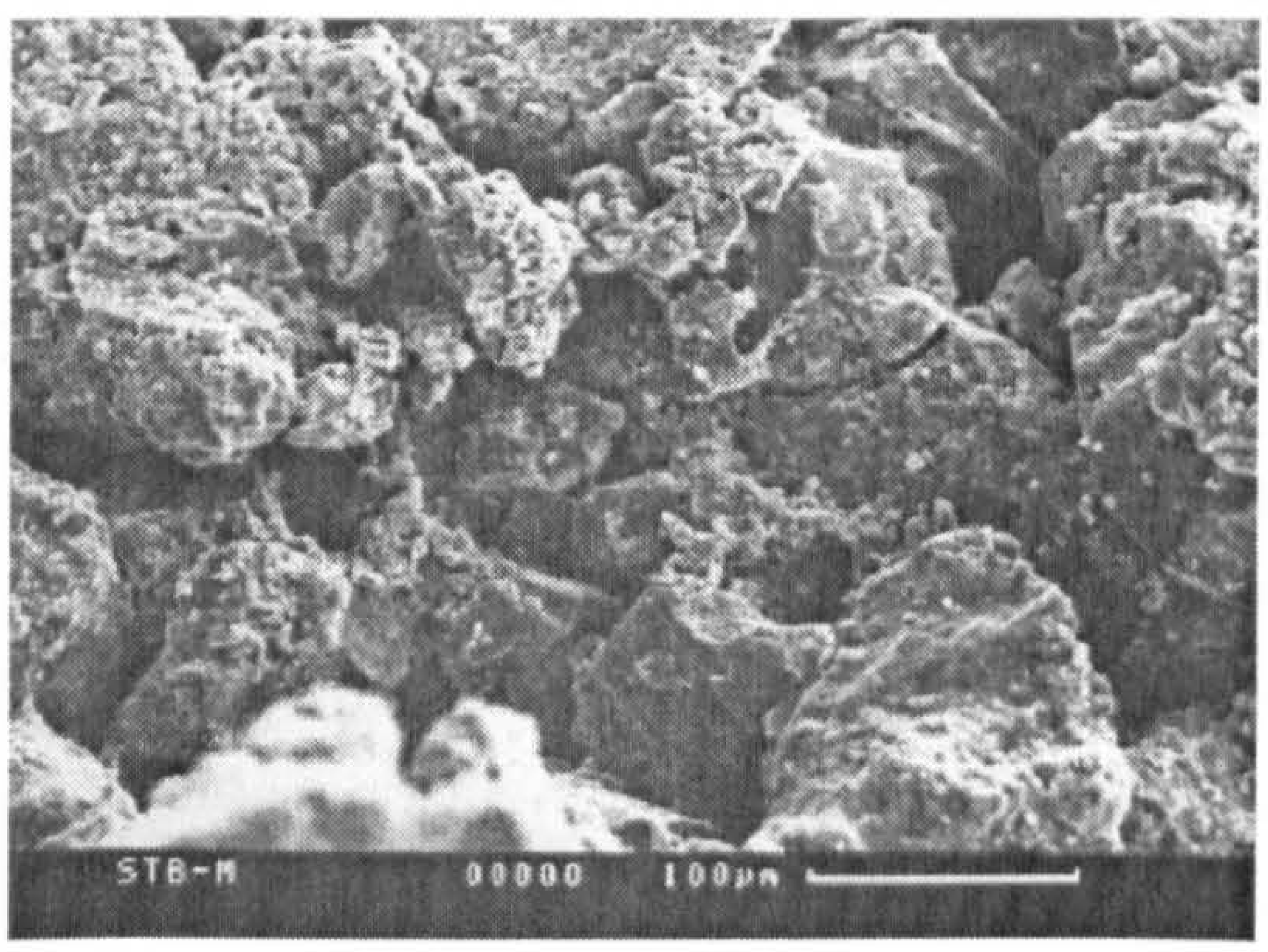
(b) Oolitic limestone: Post SW (300μm)



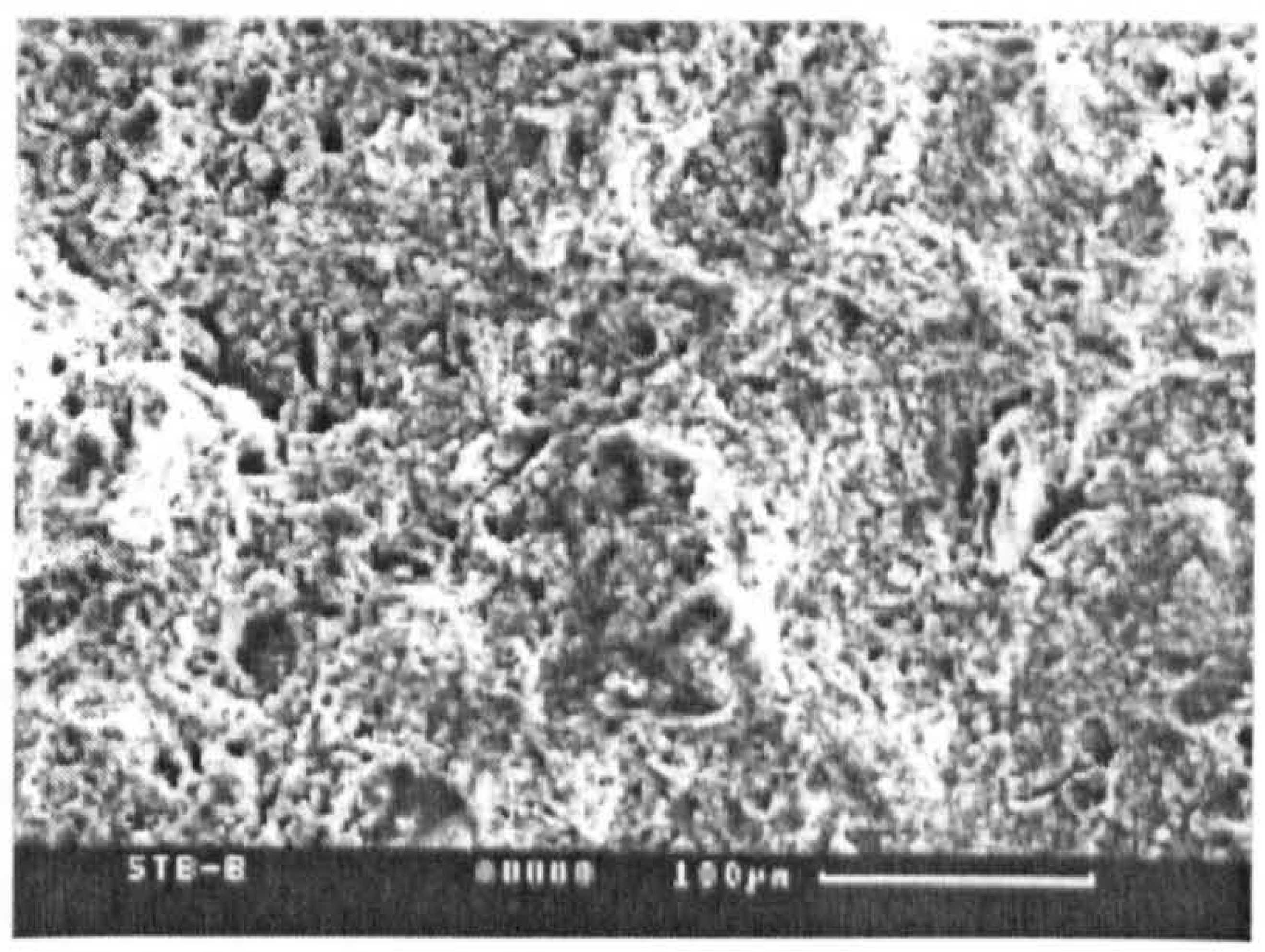
(c) Micaceous sandstone: Post SW (300μm)



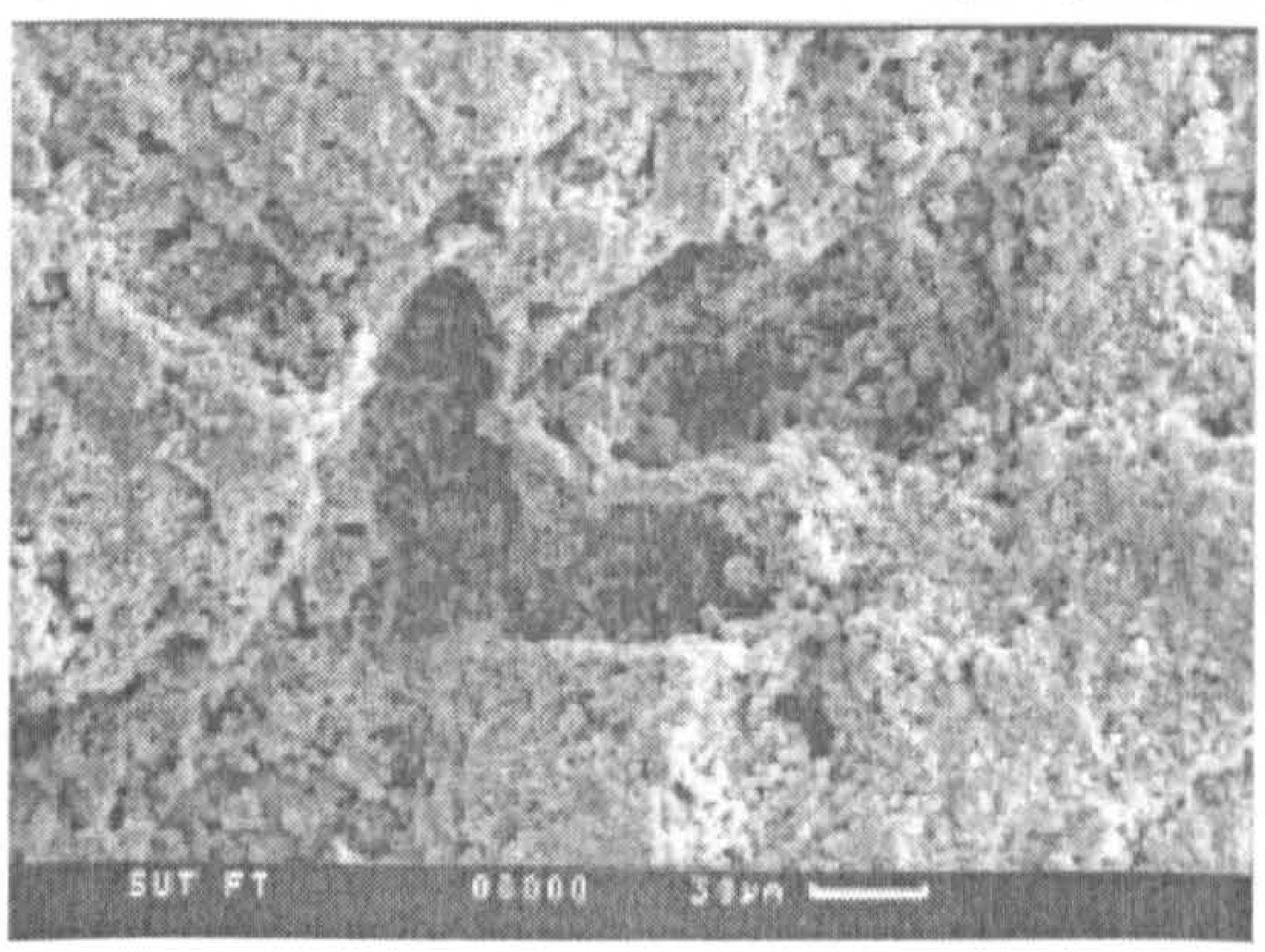
(d) Magnesian limestone: Post FT (100μm)



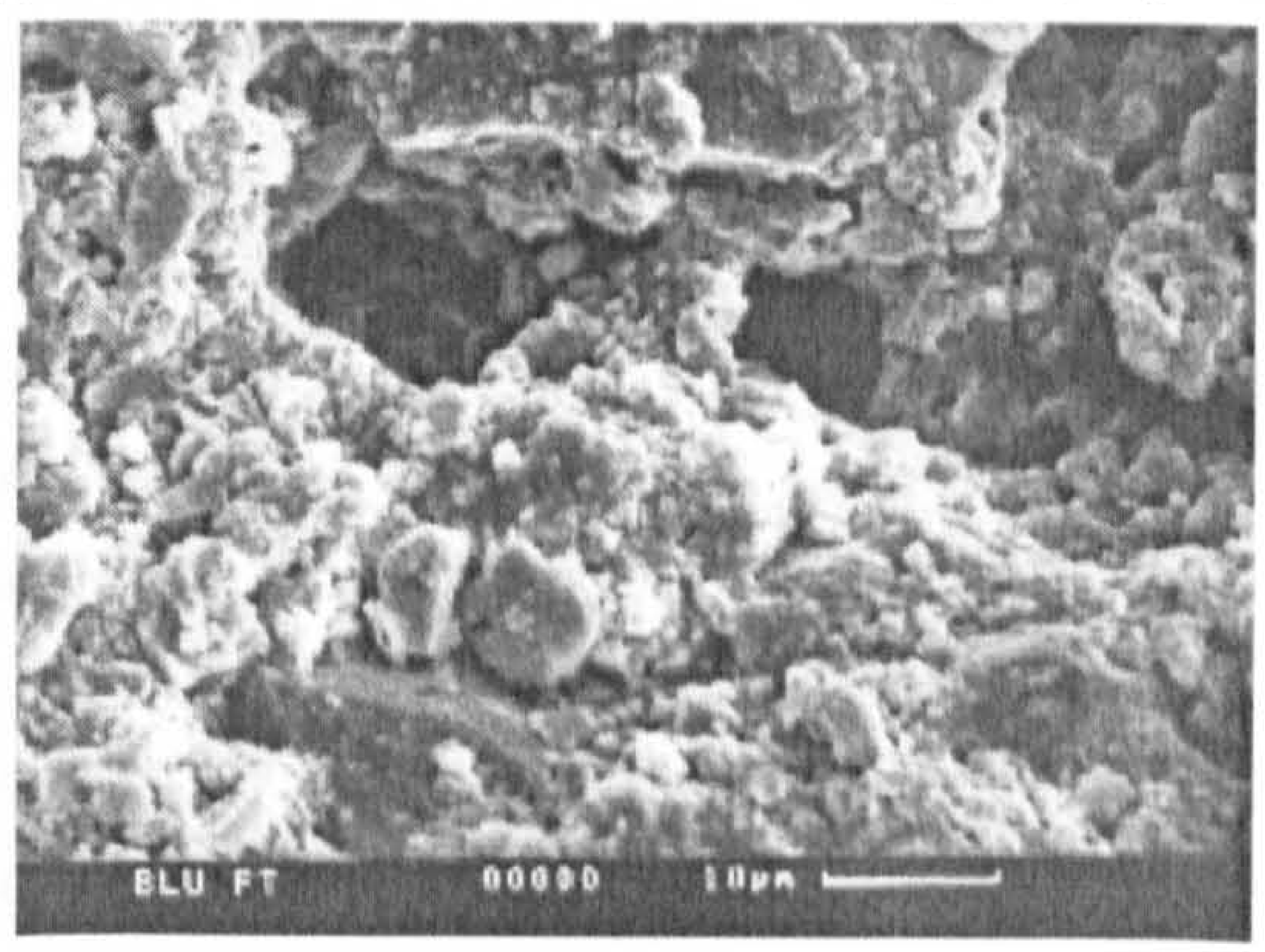
(e) Micaceous sandstone: Post FT (100μm)



(f) Micaceous sandstone: Pre-test (100μm)



(g) Calcareous sandstone: Post FT (30μm)



(h) Weathered sandstone: Post FT (10μm)

Plate 5.20 Post-test scanning electron micrographs
Refer to text for explanation (numbers in parenthesis give length of scale bar)
FT = Freeze-thaw; SW = Salt weathering

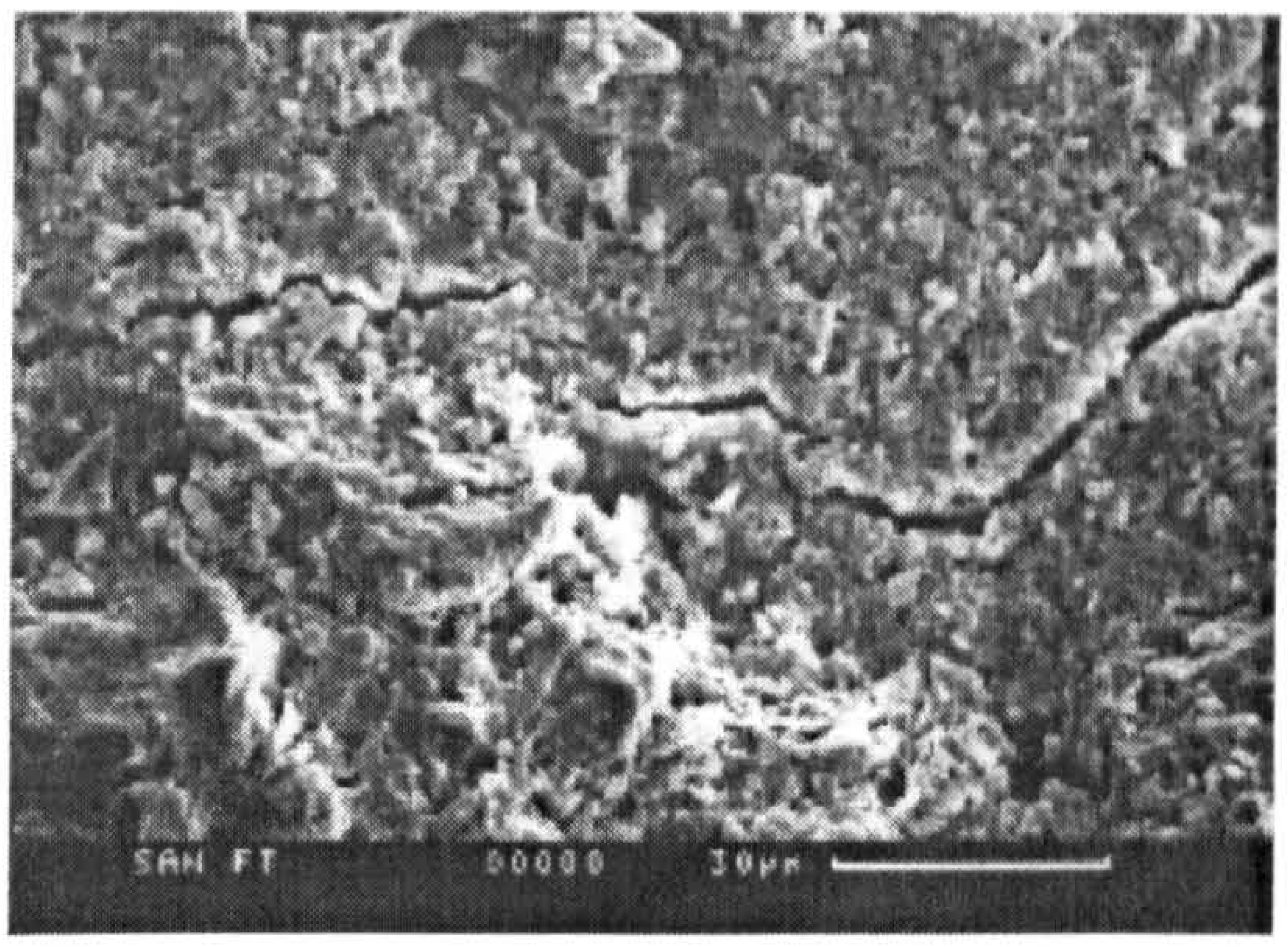
Conversely, the reduction in coarse pores could reflect similar mechanisms as described for CalS above (ie pore throat blockage due to infilling or pore compression). The proportion of pores in the middle range of 0.1 to 10μm (Plate 5.20h) might simply be the result of mechanisms already described, or could additionally reflect the introduction of new void space. SEM analysis of LamZ, for instance, reveals that a number of irregular, intergranular microcracks were generated (Plate 5.21a) due to weathering. The most likely explanation for the behaviour observed probably relates to re-distribution of debris from around pore throats. This would have the effect of clearing material from smaller pore throats and partially blocking larger throats. This results in no overall change in n_m for LamZ and a small decrease in WeaS, probably due to reduced connectivity between pores rather than any net loss of pore volume.

5.5.3.2 Salt weathering

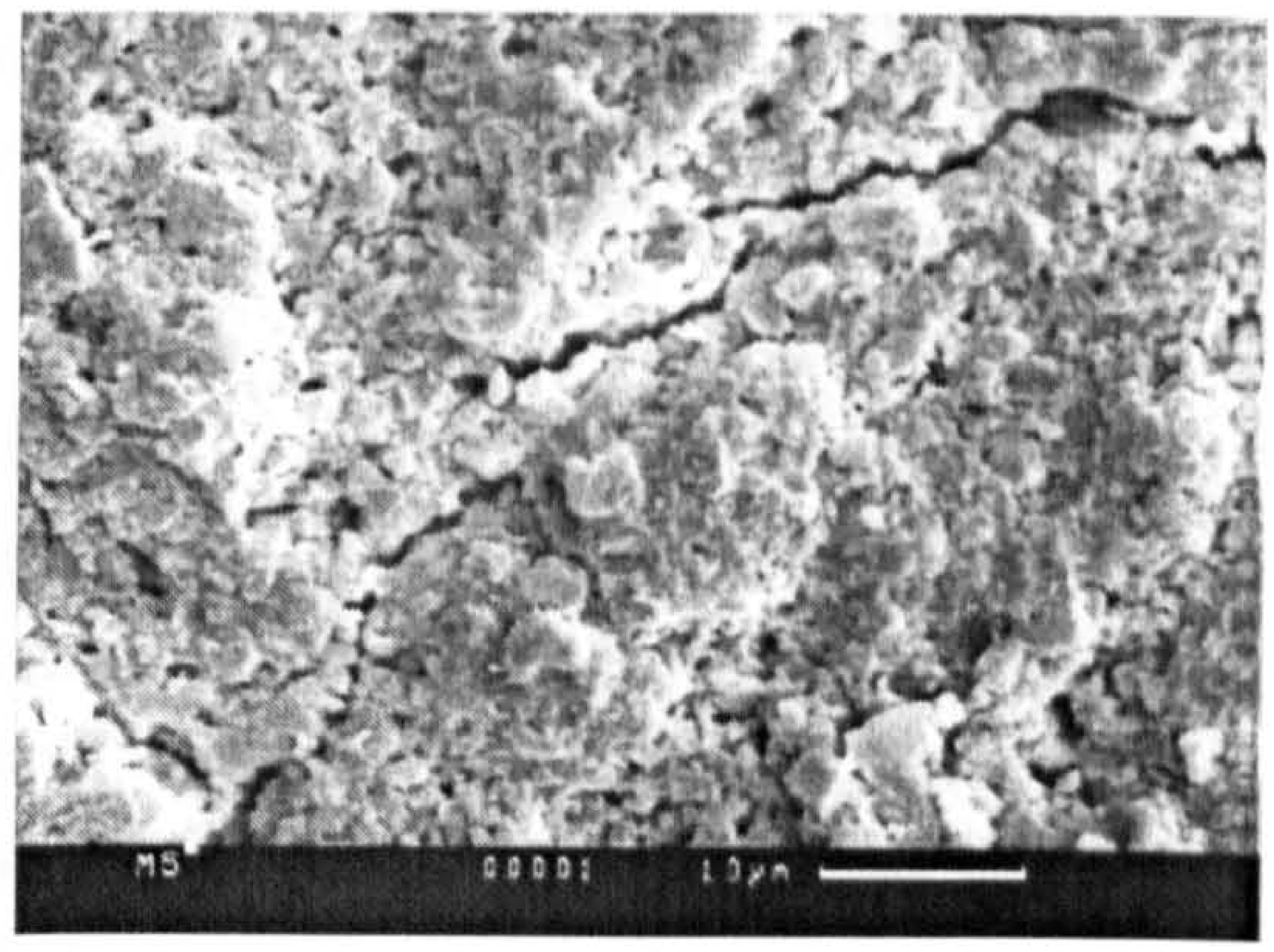
Following salt weathering, LdCh experienced a significant reduction in n_m brought about by reductions in peak pore size and the proportion of coarse pores, as well as an increase in the proportion and spread of finer pores. It is likely that this was due to ingress and crystallisation of salt in coarser pores, though there is no evidence from SEM analysis to support this. Had the weathering test resulted in the generation of any new void space, evidence could be masked by the salt deposits. CalS also showed a shift towards finer pores although primarily in the range 0.1 to 5μm. Below this, the distribution was hardly affected, and the modal pore size remained unchanged. A reduction in the proportion of pores exceeding 5μm also occurred in this rock, and there was a similar increase in μn_m as for freeze-thaw. It is likely, particularly given the small reduction in n_m that these changes were brought about by salt deposition in pores (Plate 5.20a), although some increase in void volume or connectivity might also have occurred.

A similar increase in the proportion of finer pores occurred in HdCh, suggesting again, an infilling of existing pores. In this case, however, the n_m increased by a small amount achieved by a small increase in the proportion of coarser pores. The dominance of the peak pore diameter was also considerably reduced. These changes probably represent enlargement of existing pores, or new coarse pores were generated by the weathering process. SEM analysis reveals that salt weathering induced many cracks in HdCh at all scales. As well as microcracks with an aperture of around 1μm (Plate 5.21b), there were also larger scale fractures, visible even in hand specimen (Plates 5.21c and d).

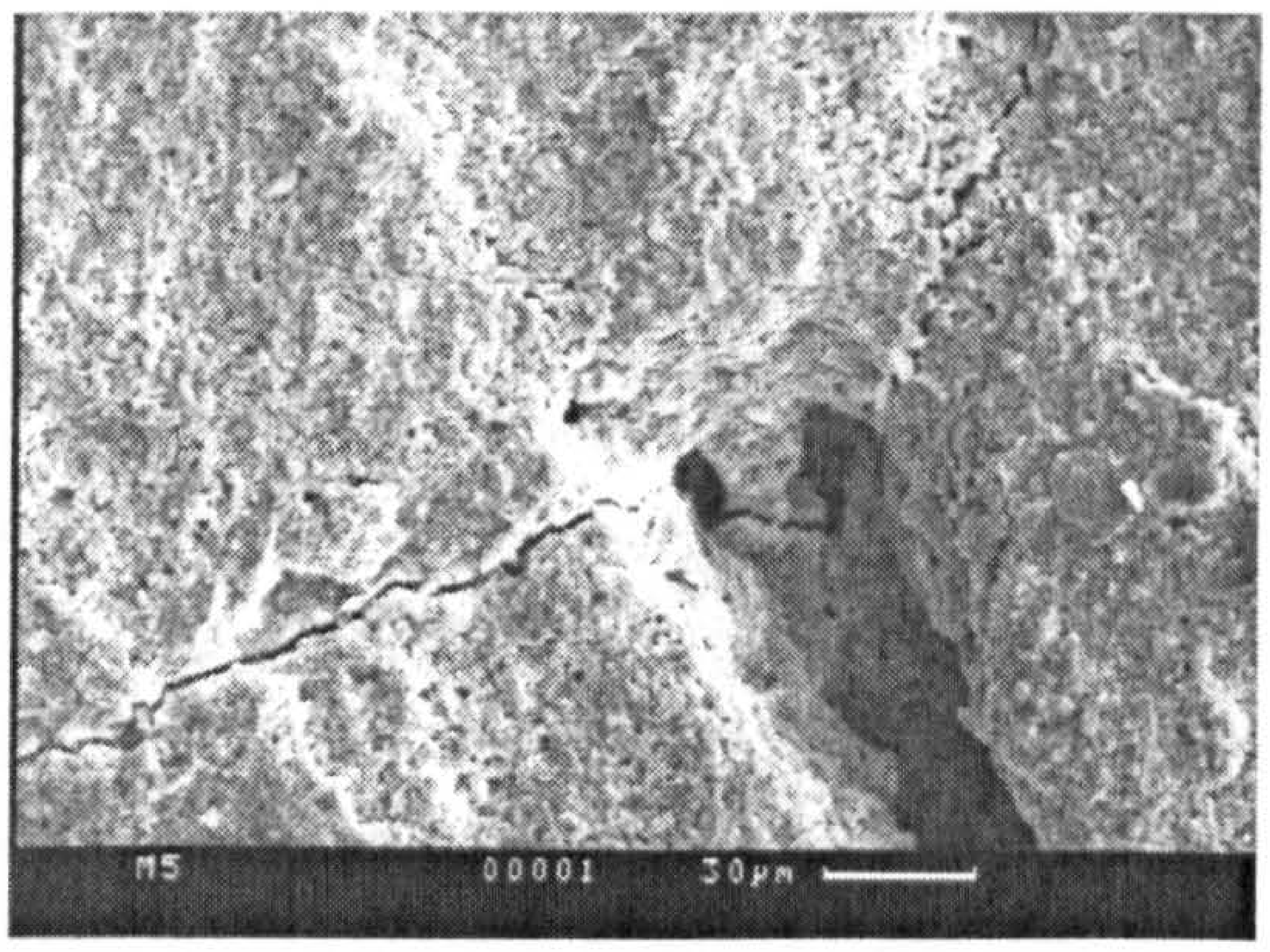
The pore size distributions of MagL and MicS responded in a similar way as for freeze-thaw. In MicS, two types of microcrack are evident from SEM analysis, namely irregular and *en echelon* cracking of authigenic grains (Plate 5.21e), and cracks linking existing pores (Plate 5.21f). These are further evidence that a real increase in n_m took place (see section 5.5.3.1). Again it is likely that for MagL the increase in n_m was achieved by a combination of improved connectivity and generation of new void space. However, the pore size distribution also indicates that the proportion of pores in the range 1-10μm has been slightly reduced, probably by infilling contributing to the increase in pores in the range 0.1 to 1μm. Pore infilling apparently did not occur to the same extent in MagL as for the other limestones. Studies by others (eg Fitzner 1988; Ordonez et al 1997) indicate that the initial deposition and growth of salts and ice crystals occurs first in large pores. Once these are filled further growth continues into smaller diameter



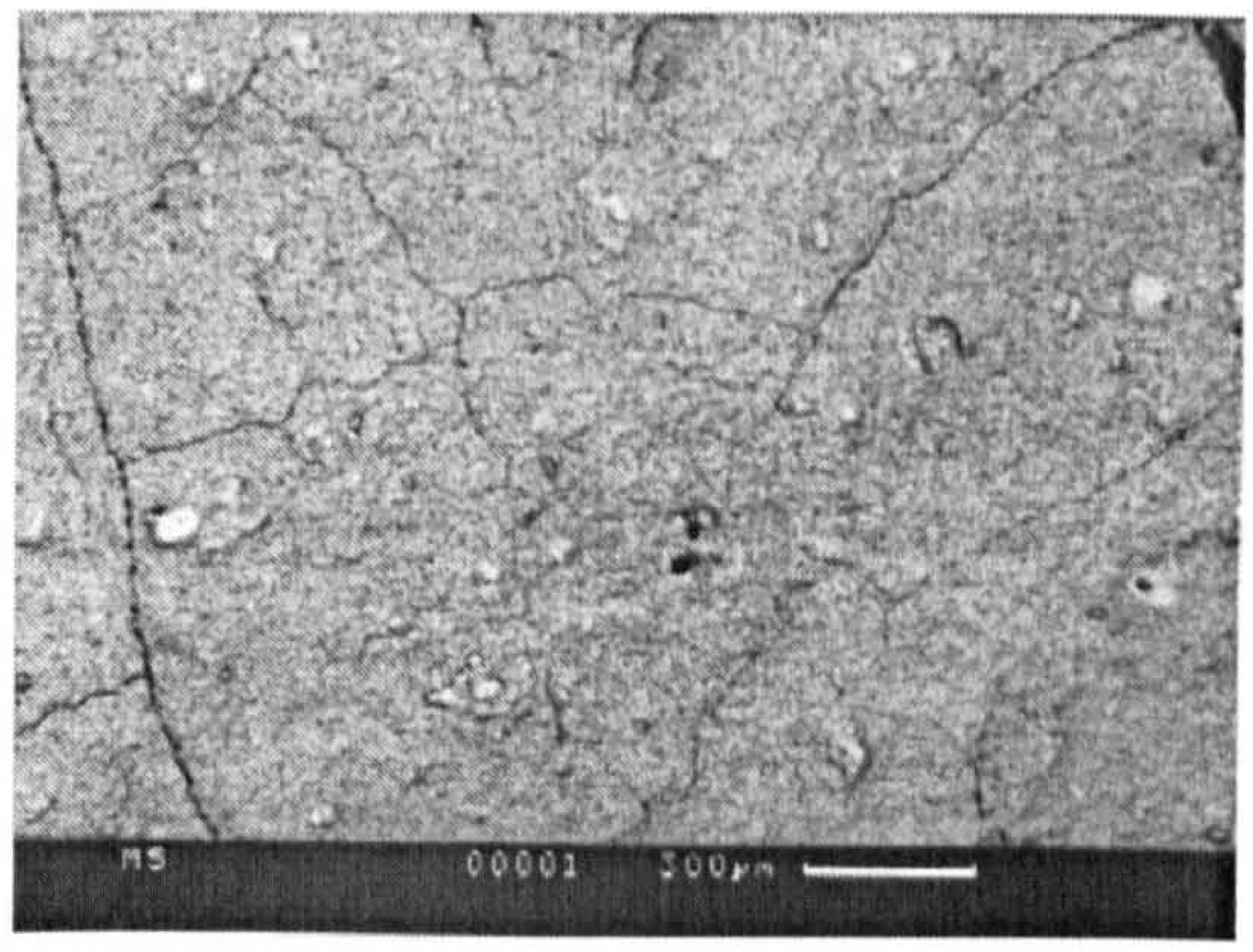
(a) Laminated siltstone: Post FT (30μm)



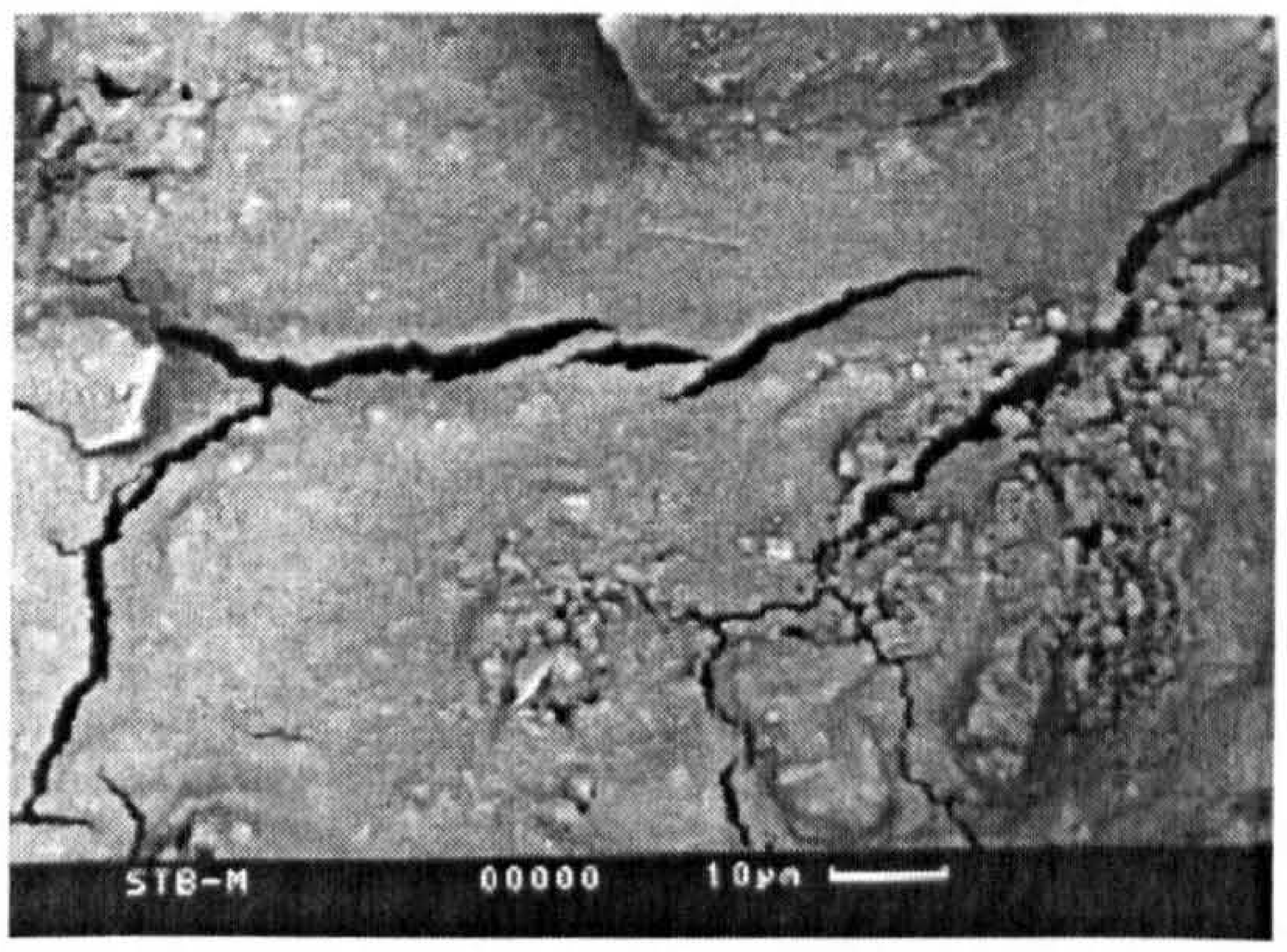
(b) High density chalk: Post SW (10μm)



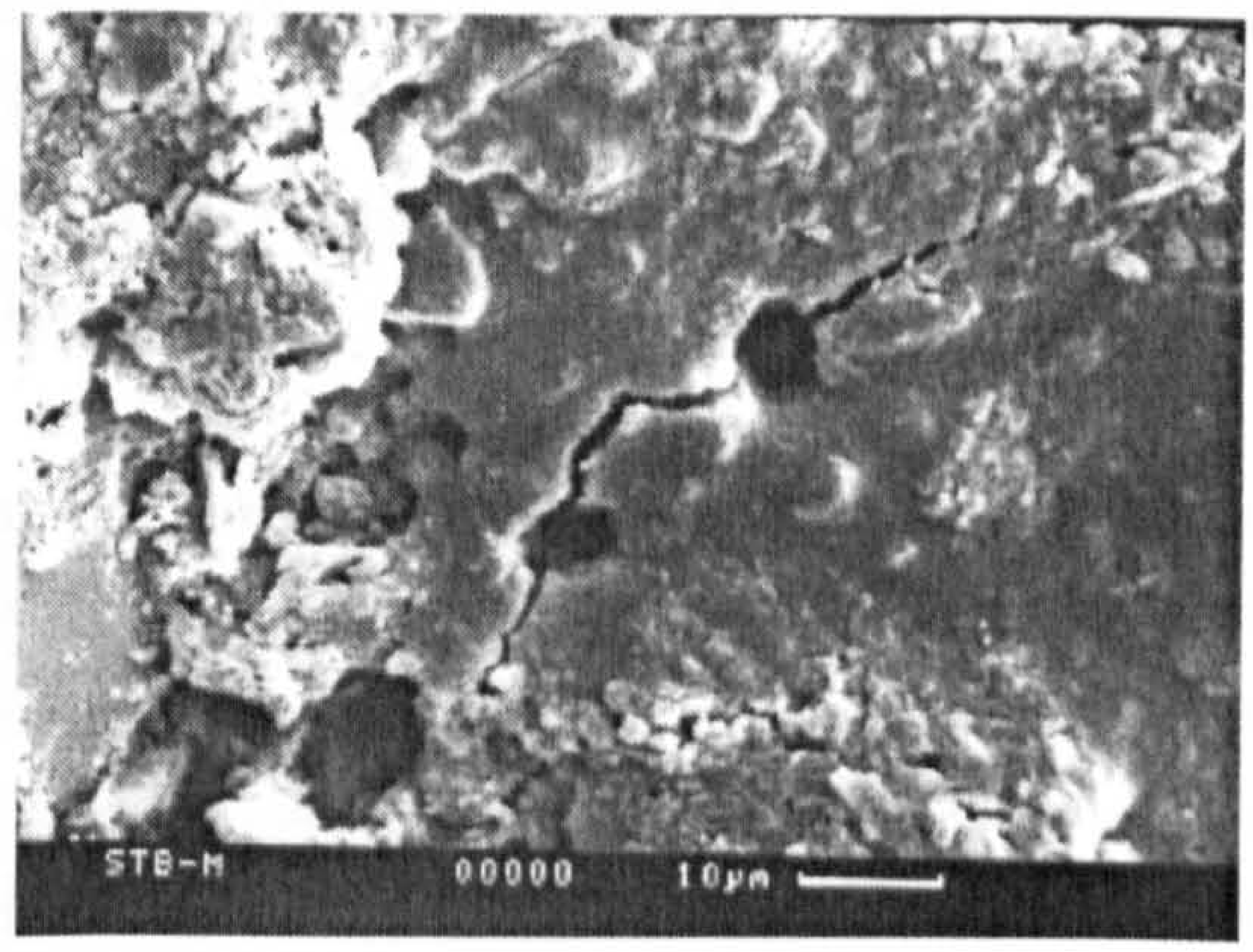
(c) High density chalk: Post SW (30μm)



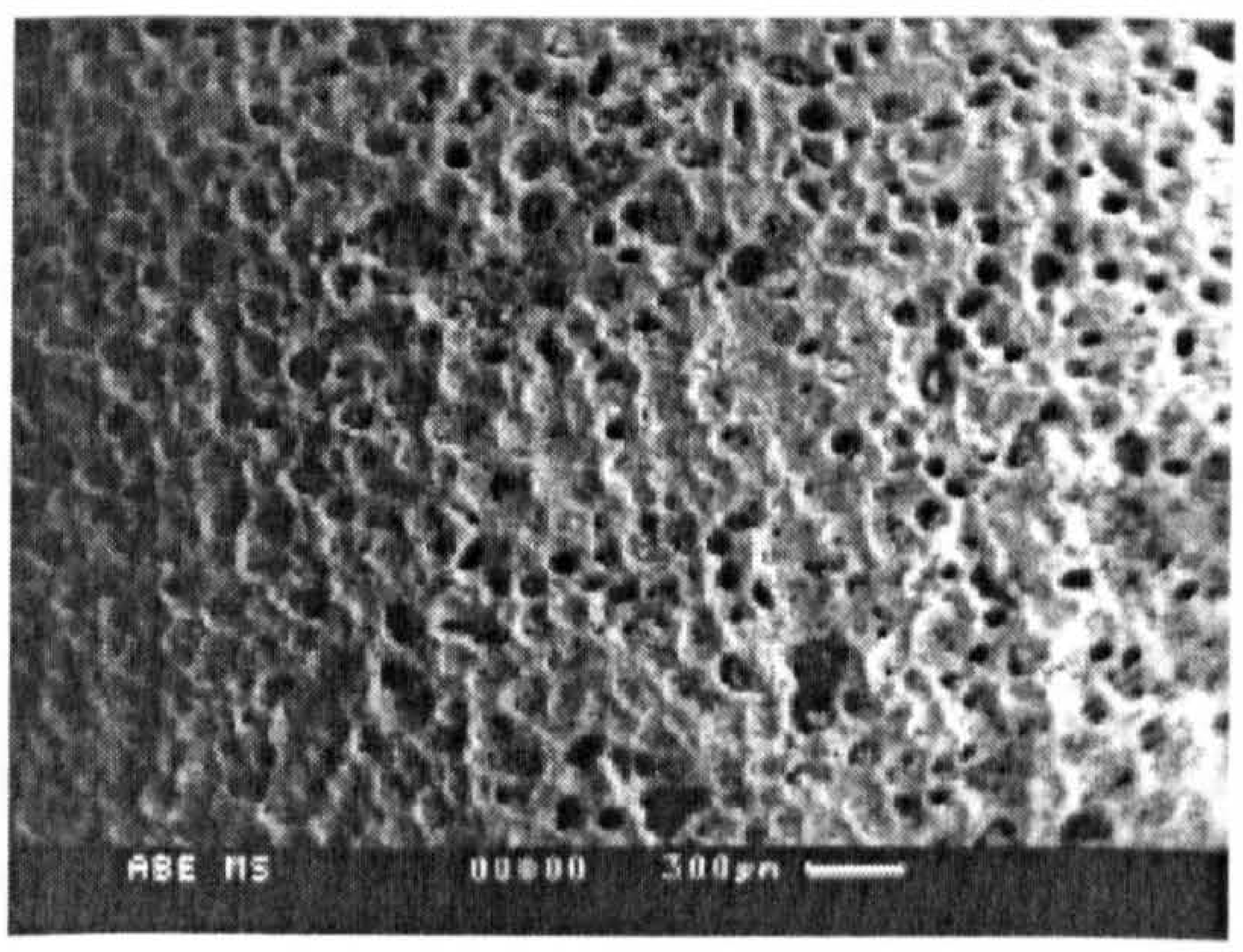
(d) High density chalk: Post SW (300μm)



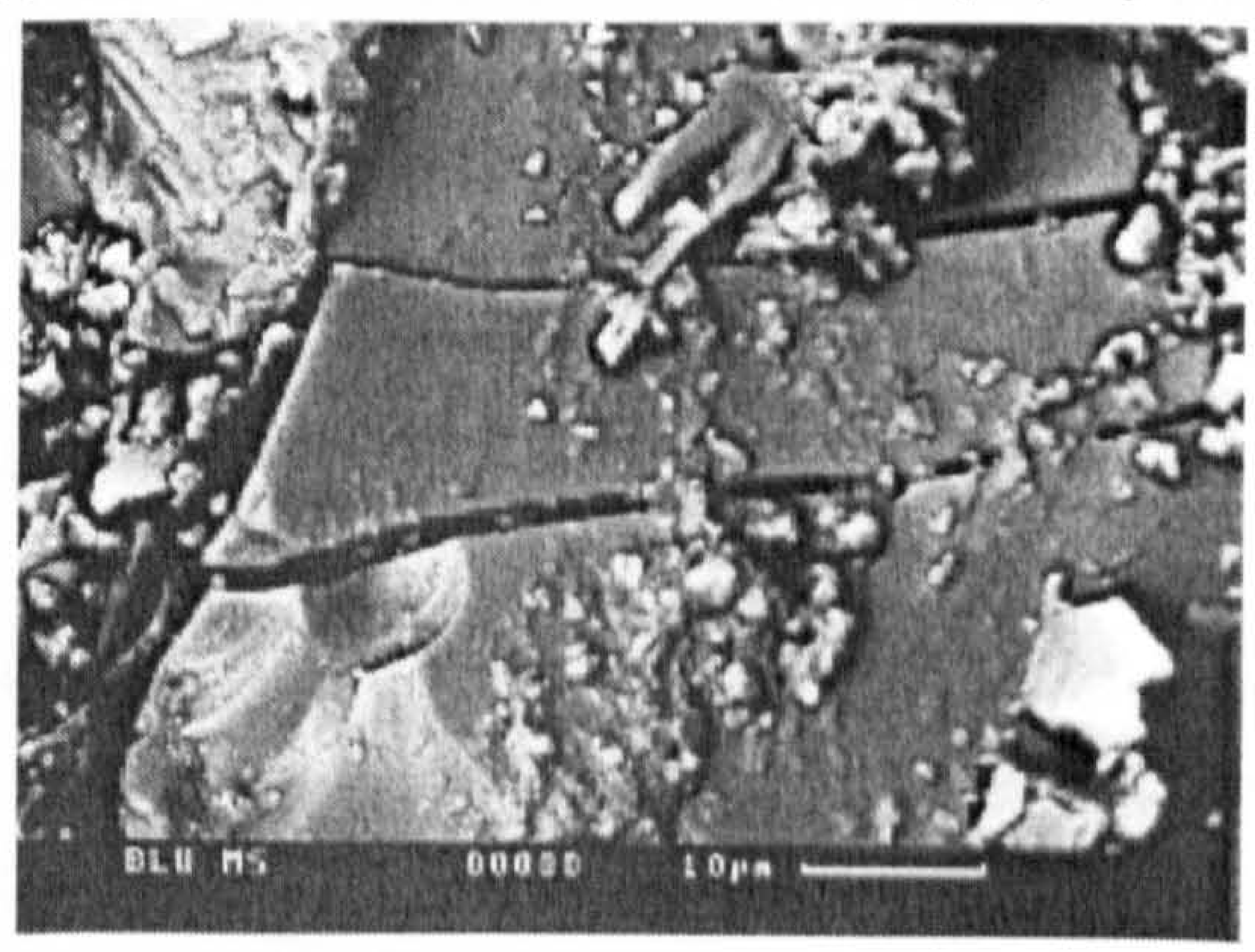
(e) Micaceous sandstone: Post SW (10μm)



(f) Micaceous sandstone: Post SW (10μm)



(g) Magnesian limestone: Post SW (300μm)



(h) Weathered sandstone: Post SW (10μm)

Plate 5.21 Post-test scanning electron micrographs

Refer to text for explanation (numbers in parenthesis give length of scale bar)

FT = Freeze-thaw; SW = Salt weathering

pores. MagL has a notably higher proportion of coarse pores than the other limestones (Plate 5.21g) and it is possible that salt did not enter many of the finer pores since the coarse pores had not been completely filled.

For OolL the overall distribution of pore sizes was very similar to its pre-test form, with the increase in n_m being accommodated by an increase in the proportion of pores of all sizes by introduction of new void space. This indicates that there was some fundamental lithological control on pore structure such as grain size distribution and packing or the nature of grain contacts. There was less change in n_m than that measured for freeze-thaw probably due to pore infilling.

As with the freeze-thaw test, there were similarities in the pore size distributions for WeaS and LamZ following salt weathering. For pores up to 0.1µm there was a shift towards coarser pores which was transitional between the pre-test and post-freeze-thaw distributions and coalescence and/or enlargement of fine pores might be responsible for this. In the middle range of pores from 0.1 to 10µm both rocks showed an increase in pores over pre-test proportions. SEM analysis suggests that in WeaS this might be due to microcracking, particularly in association with authigenic grains (Plate 5.21h). In LamZ, these medium pores made a significantly larger contribution to overall porosity than following freeze-thaw. It is at the coarse end of the range of pores where the two samples differ. For WeaS, the proportion of pores >10µm was considerably less after testing and comparable to the distribution following freeze-thaw and probably indicates pore infilling. For LamZ, the proportion of pores >10µm increased dramatically forming a new and dominant peak pore diameter of around 22µm. This probably indicates generation of new void space, in part, as a result of extensive microcracking.

The porosity of SpaL and MetS was below the resolution of the measurement method and thus pore size distributions and their pre and post-trends are grossly exaggerated. Following salt weathering, mercury was unable to penetrate pores smaller than 10µm in SpaL, and this might be due to salt infilling of pores though there is no SEM evidence to support this. SEM analysis also reveals limited microcracking but this is not distinctly different from that which was evident prior to testing. For MetS it would appear that new, finer void space was introduced though this is not supported by SEM analysis. An alternate explanation is that mercury was able to gain greater access into the interior of the rock and hence to these pre-existing, fine pores, via newly developed macro surface cracks.

5.5.3.3 Comparison of pore size distribution and n_e

There is generally good agreement between implied pore modification derived from n_e and pore size distribution data. There are a number of cases where n_e indicates no change in pore structure where some change is suggested by post-test pore size distribution. This is to be expected since high pressure mercury intrusion gives data which represents maximum effective porosity and therefore the pore size distribution derived from this test gives a much fuller representation of all pore sizes than is possible with ambient pressure saturation.

5.5.4 Relationship between inferred pore modifications and rock deterioration

A basic hypothesis of this work is that rock breakdown at the macro scale by fracturing and fragmentation will be preceded by modification at the micro scale, including microcracking, pore coalescence and enlargement, leading to the generation of macro fractures and fragmentation. In granular rocks, grain boundary microcracks (Simmons and Richter 1976) can lead directly to disintegration and this has been seen to a limited extent in OolL. In non-granular rocks, however, microcrack linking and coalescence can give rise to macrofractures which are visible at the rock surface. If sufficiently persistent, these give rise, in turn, to detachment and fragmentation. Pores can also contribute to this process by enlargement and coalescence. It is reasonable to expect, therefore, that macro-scale signs of deterioration would be reflected in micro-scale modifications of the type described above. In the next section, therefore, the relationships between the inferred pore modifications and rock deterioration are explored. For convenience, rocks are grouped with respect to their deterioration response.

5.5.4.1 Rocks which deteriorated

For most rocks which did deteriorate, there were clear signs of pore modification occurring. Rocks which showed significant deterioration and also pore modification such as improved connectivity and new void space were MagL, OolL, HdCh and LamZ for the freeze-thaw test, LdCh, MagL, HdCh, WeaS, CalS, MicS and LamZ for the salt weathering test, and LamZ for both the wetting and drying and slake durability tests. For freeze-thaw, CalS also showed an increase in μn_m but no significant change in n_m, suggesting a re-distribution of debris. LdCh, which deteriorated severely after freeze-thaw, unusually showed a significant reduction in n_m and is discussed separately in section 5.5.4.3.

Although some samples followed similar trends in terms of both deterioration and pore modification occurring, they might, nevertheless, differ markedly in terms of the absolute amount of deterioration which occurred. This is particularly the case for the freeze-thaw test. For example, HdCh experienced almost four times the weight loss experienced by MagL, yet both were characterised by an increase in n_e indicating improved connectivity. These differences could relate to other textural or lithological controls. The fact that the total pore volume of MagL is much higher than that of OolL, for instance, and that its saturation coefficient is much lower, might represent a greater capacity to take up the pressure of ice crystallisation. Alternatively, it could be explained by the partially interlocking nature of MagL compared to the relatively weakly cemented OolL.

Although most of the rocks which experienced deterioration due to salt weathering also experienced pore infilling, damage was sufficiently great that pore modifications were also detectable in some cases. This is the case for MagL, HdCh and WeaS. It is interesting to note that the style of deterioration for these rocks differed markedly. Whereas MagL and HdCh were subject to moderate and intense fracturing, WeaS deteriorated primarily by minor fragmentation and grain loss. In each case, however, pore size distribution analysis reveals that new void space was introduced as a result.

Analyses of changes in post-test pore size distribution, n_e and micro-structure (using SEM) indicates that rocks in which *new void space* was generated are also those which suffered the most intense deterioration due to weathering (eg OolL and LamZ after freeze-thaw; HdCh, WeaS and LamZ after salt weathering; LamZ after wetting and drying and slake durability), though LdCh is anomalous in this respect (section 5.5.4.3). Conversely, of the rocks which did deteriorate, those in which *improved connections between pores* were inferred are also those which seemed unusually resistant to weathering (eg MagL and HdCh after freeze-thaw).

5.5.4.2 Rocks which resisted deterioration

It is to be expected that rocks which largely resisted deterioration not show change or significant change in pore properties. This was the case for SpaL and MetS for the freeze-thaw and salt weathering tests, LdCh and CalS for wetting and drying, and MagL, OolL, SpaL, WeaS, MicS and MetS for the slake durability test. The nature of the weathering process involved in slaking is such that breakdown is by surficial abrasion and granular disintegration. As such, it is not expected that significant modifications to the pore structure would occur by microcracking for instance, although pore enlargement by dissolution and improvement to pore connectivity are still possible. Using the empirical approach of Winslow and Lovell (1981) in which the slake durability index is calculated on the basis of a range of pore properties, completely anomalous results are obtained for the rocks tested here. In part, this is an indication that for this group of rocks slaking cannot be expected to produce changes in pore structure.

It is notable that modification of the existing pore structure, in the form of increased connectivity, could be inferred in several samples which did not deteriorate (eg HdCh after wetting and drying; HdCh and CalS after slake durability) or which experienced minimal deterioration (LdCh and CalS after wetting and drying). This supports the assertion made earlier that stresses and cyclic changes induced by weathering would bring about a rationalisation of the existing pore structure before any *new void space* was generated.

5.5.4.3 Anomalous results

There are several cases for which observed changes are anomalous in that pore modifications do not seem to be in accord with the deterioration observed.

(i) The Low density chalk (LdCh)

Following freeze-thaw LdCh showed a reduction in n_m, modal pore size and μn_m. This was not expected, given the severe deterioration which occurred for this rock. It is feasible to suggest, given the inherent weakness and high compressibility of LdCh, that a reduction of void volume and connectivity could have occurred as a result of compaction or pore collapse. Re-distribution of debris or case hardening are further possibilities which could explain these results. Conversely, after slaking, in which significant rounding of LdCh specimens occurred, no pore modifications were apparent. This probably reflects the nature of the test in which surface mechanical abrasion and damage is much more important than internal rupture.

(ii) Rocks which resisted deterioration but which showed pore modification

High density chalk (HdCh): The high density chalk strongly resisted deterioration following wetting and drying and slake durability but pore modification occurred in both cases. Wetting and drying induced a significant increase in n_e and improved saturation efficiency rather than an increase in total void volume can be inferred. The increase in n_e occurred steadily and progressively from the start of testing without any reciprocal change in ρ. It is unclear whether by the end of testing the observed change in n_e represents a stable condition or whether visible deterioration would have occurred had there been further cycles of wetting and drying. The reduction in n_e was similar to that experienced by CalS for slake durability and might indicate case hardening or slight compaction of the rock due to impact and abrasion. This seems unlikely for HdCh and CalS, the two stronger rocks, particularly since it is an effect which is absent in other weaker rocks (eg MagL and OolL). A further possibility to explain the behaviour in the slaking test of HdCh and CalS is that near-surface clogging of pores due to fines in suspension in the slaking water could have reduced water absorption at the surface.

Weathered and micaceous sandstones (WeaS and MicS): The weathered and micaceous sandstones were highly resistant to weathering from the freeze-thaw test (as indicated by data given in Table 4.1) yet experienced a pore modification. WeaS showed a reduction in n_m and a significant increase in μn_m. Given the relatively coarse nature of the pore structure this was probably a reflection of the ease of water ingress and egress and its effect in re-distribution of fines contained within, of which this rock contained a high proportion. MicS showed a significant increase in n_m and a reduction in μn_m. The possible reasons for this were discussed in section 5.5.3.1.

Oolitic limestone (OolL): The oolitic limestone largely resisted deterioration due to salt weathering, yet showed a significant increase in n_m (refer to possible explanation given in section 4.4.4).

5.5.4.4 Further discussion

Analysis of pore modifications due to weathering indicates that some effects owe more to the nature of test conditions than to breakdown mechanisms per se (eg pore infilling by salt). These are more difficult to interpret in terms of their role in deterioration. It is also clear from the results here that for some rocks, deterioration in the form of fracturing or weight loss occurs in the absence of any apparent pore structure modification. This is despite the assertion in section 5.5.4 above that macro change might be discernible at the micro scale. There are several situations in which this can be envisaged:

(i) Granular loss of the type which occurred in MicS is essentially a surficial deterioration mechanism involving detachment of grains via grain boundary cracks, and their subsequent removal. There is no reason to believe that such a process should affect internal pore structure. This might also apply to rock deterioration induced by slaking.

(ii) Fracturing of the type which occurred in MetS might be non-penetrative, or the fractures so tightly closed that no modification of internal structure occurs. Alternately, if microcracks propagate and coalesce instantaneously to form macrocracks there might be no evidence of modifications to pore structure.

(iii) Pore modification might be masked by other processes such as infilling by salts.

However, in many cases, pore modifications were apparent for rocks which deteriorated and this is evidence of a link between micro and macro deterioration processes. There is also evidence that pore structure modification occurs in a progression through three distinct stages:

A: Increased pore connectivity by *modification of existing pore structure*:

The first stage in this progression is pore structure modification which leads to an increase in water absorption but does not result in any change in the total volume of void. Less energy expenditure would be involved if internal weathering-related pressures from water migration or crystallisation of ice or salt, for example, could be taken up in existing pore spaces rather than creating new void. This could be achieved by the breaking up of grain contacts and re-distribution of debris. These mechanisms are likely to modify the pore size distribution and increase connectivity, but are not accompanied by any visible or measurable deterioration at the macro scale. The response of HdCh to wetting and drying is an example of this behaviour.

B: Increased total void volume by *modification of existing pores*:

The second stage in this progression involves an increase in the total volume of void in rock, achieved by modification, particularly enlargement, of existing pores (eg by debris re-distribution or dissolution). The response of MagL to freeze-thaw is an example of this behaviour.

C: Increased total void volume by *generation of new void*:

In the final stage, new void is generated by microcracking, or by pore coalescence and linking. The response of OolL to salt weathering is an example of this behaviour.

It is likely that as internal pore structure modifications progress from stage one to three, macro deterioration at the surface would increasingly become apparent. The mode and severity of such deterioration might be broadly unrelated to pore structure however, and as stated before, more dependent upon a range of mechanical, lithological and structural rock properties.

5.5.5 Mechanical strength (freeze-thaw only)

For the freeze-thaw test, Figure 5.27 shows the percentage change in point load strength (IS_{50}) for each rock type and modulus of rupture (T_{mr}) for HdCh and LamZ. Summary data are also given in Table 5.4. For most samples (LdCh, MagL, OolL, HdCh, HdCh(T_{mr}), WeaS, LamZ and LamZ(T_{mr})), a net reduction in strength occurred with percentage change in IS_{50} ranging from around -13% for HdCh to -41% for OolL. Reductions in T_{mr} (shown in red) were more substantial with -69% for HdCh and -77% for LamZ. Two rocks, MicS and MetS, showed a small net increase in IS_{50} though the data for MicS were based on a very small sample and might not, therefore, be very reliable. Furthermore, the standard deviation of point load strength for MetS was considerably higher than for the other rocks and so it is likely that the apparent increase in strength might simply reflect sample heterogeneity. There was negligible change in strength for CalS and SpaL. Several samples showed significant fluctuation in strength during the test, with increases of 7 to 13% occurring early on in LdCh, MagL and MicS, and decreases of 13% and 12% occurring in CalS and SpaL.

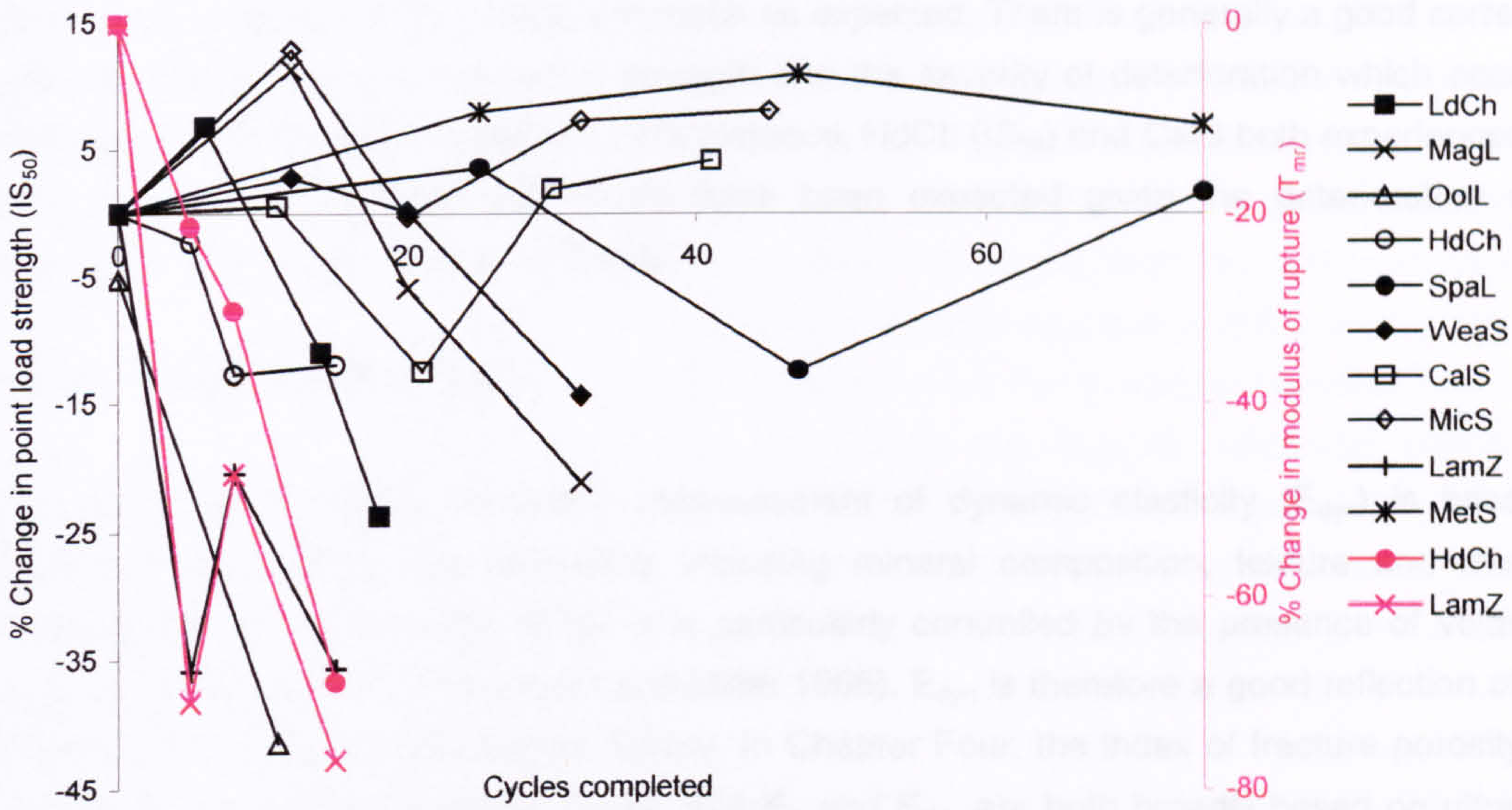


Figure 5.27 Percentage change in rock strength during freeze-thaw

IS_{50} (MPa)		LdCh	MagL	OolL	HdCh	SpaL	WeaS	WeaSw*	CalS	MicS	LamZ	MetS
Pre-test	Mean	0.8	0.6	1.00	3.1	5.2	1.5	2.6	2.6	2.3	5.6	11.8
	Number of specimens	10	8	10	8	8	7	10	10	10	11	8
Post-test	Mean	0.6	0.5	0.6	2.7	5.3	1.3	2.7	2.7	2.5	3.6	12.5
	Number of specimens	6	6	3	14	8	8	8	8	4	9	8
	Cycles completed	18	32	11	15	75	32	41	41	45	15	75
T_{mr} (MPa)		LdCh	MagL	OolL	HdCh	SpaL	WeaS	WeaSw*	CalS	MicS	LamZ	MetS
Pre-test	Mean				8.7						19.0	
	Number of specimens				9						8	
Post-test	Mean				6.1						10.0	
	Number of specimens				5						6	
	Cycles completed				8						8	

Table 5.4 Pre and post- freeze-thaw test data for IS_{50} and T_{mr}

Note WeaSw were sandstone cubes cut from a block considerably more weathered than the main sample of WeaS

Rock strength is a function of several properties including the hardness of mineral constituents, the degree of packing, grain sorting, texture and the nature of the intergranular bonding material. Moisture content is also critical but since samples were tested here in an oven-dry state this can be ignored here. These properties are closely related to density and porosity and it can, therefore, be expected that they might be modified due to weathering. Intra-granular microcracks, dissolution and alteration (eg by stress corrosion), for instance, might weaken mineral constituents leading to a reduction in density. Inter-granular weakening could also occur by development of grain boundary microcracks and the breaking up of grain contacts. These in turn, could loosen the grain packing and increase porosity. As seen above, porosity can also be increased due to dissolution.

Given these potential changes it is not surprising that strength reductions are evident following freeze-thaw testing and the results are much as expected. There is generally a good correlation between the amount of reduction in strength and the severity of deterioration which occurred. However, there are some exceptions. For instance, HdCh (IS_{50}) and CalS both experienced less of a reduction in strength than might have been expected given the deterioration which occurred. The opposite is true for WeaS.

5.5.6 Modulus of elasticity

Ultrasonic pulse velocity, on which measurement of dynamic elasticity (E_{dyn}) is based, is influenced by several rock properties including mineral composition, texture and moisture content (Attewell and Farmer 1976). It is particularly controlled by the presence of voids and anisotropy (Augustinus 1991, Deere and Miller 1966). E_{dyn} is therefore a good reflection of rock strength and stiffness (see Chapter Three). In Chapter Four, the index of fracture porosity was used as a deterioration indicator. Given that IF_p and E_{dyn} are both broadly based on ultrasonic velocity it could be expected that trends for these two indices would be identical, albeit with different absolute data values. Although there are some similarities between the two sets of results, this is not the case. This is because IF_p is a measure of the void volume introduced by the weathering process and is simply a function of pre- and post-test P-wave velocity (Vp) over a given specimen length. On the other hand, E_{dyn} is a function of three measures, Vp, Vs and ρ (see section 3.4.3.3). It thus reflects the capacity of the rock for lateral as well as axial strain and because Vs is also measured, is likely to be less erroneous due to anisotropy. This is because heterogeneities and anisotropy have a larger effect on shear waves than they do on compressional waves in terms of attenuation (Lucet and Zinszner 1992).

The percentage change in elasticity has been used by a number of authors as a gauge for rock durability (Fahey and Gowan 1979; Allison 1988, 1990; Allison and Bristow 1999; Allison and Goudie 1994; Goudie et al 1992; Murphy and Inkpen 1996). Percentage change in E_{dyn} for each test is given in Figures 5.28, 5.29 and 5.30, and summary data are presented in Table 5.5. Comparison with fracture porosity charts presented in Chapter Four (Figures 4.2e; 4.3c and f; 4.4c and f; 4.5c, f and i; 4.6c and f; 4.7c and f; 4.8c, f and h; 4.9c and f; 4.10c and h; and 4.11c and f) shows that for the most part, general trends are similar (though an increase in IF_p, of course, translates to a decrease in E_{dyn}). E_{dyn} data is notably more consistent, though, with less variability between specimens, and less erratic temporal behaviour than for IF_p. This is an indication that percentage change in E_{dyn} is perhaps more robust than IF_p, since, being based on both P and S-waves, is more representative of the whole rock including any heterogeneities.

5.5.6.1 Freeze-thaw test

Some samples (OolL(2), HdCh, MicS, LamZ) showed a reduction in elasticity following freeze-thaw. A small reduction in E_{dyn} also occurred for WeaS but an anomalous specimen gives a mean increase. An overall *increase* in E_{dyn} was recorded for MagL, though this followed a very significant fluctuation after 12 cycles. Being a particularly weak material, it is possible that freeze-thaw induced damage to MagL early on in the test procedure. This could have produced microcracking, pore enlargement and the breaking up of grain contacts, for instance. Application of the 34kPa load during measurement of ultrasonic pulse velocity (see section 3.4.3.4) after 12 cycles might have then caused the closure of new and existing voids, giving an apparent increase in rock quality. Work by New (1976) and New and West (1980), however, indicates that acoustic closure would not occur at such low normal stresses and that in fact a normal stress of at least 0.1MPa would be required to achieve a consistent P-wave velocity. However, their work utilised saw-cut discontinuities which persisted across the axis of cylindrical specimens. Ultrasonic velocity across these saw-cut discontinuities was found to increase very significantly even by application of very small stresses, although maximum *consistent* values for Vp were not recorded until acoustic coupling had occurred at much higher levels of stress, around 0.1MPa for strong igneous rocks, 0.4MPa for chalk and 0.75MPa for sandstone (New 1976).

	LdCh	MagL	OolL(2)	HdCh	SpaL	WeaS	CalS	MicS	LamZ	MetS
Freeze-thaw	-	8.91	-15.49*	-28.79	0.17	1.52	-1.84	-16.52	-73.71	5.01
Salt weathering	-	94.86	22.20	-43.96	10.28	46.48	17.20	9.46	-	-6.96
Wetting and drying	-4.24			-2.22			-0.45		-80.42	

Table 5.5 Percentage change in E_{dyn} for each weathering test

Note Based on a single specimen OolL(2)

It is proposed that in considering the much smaller, non-persistent, natural, and therefore closely matching internal fractures likely to have been present in MagL, some degree of acoustic closure could have occurred even with the relatively low stress applied (0.034MPa). This is supported by the reduction in n_e which was also recorded after 12 cycles (Figure 5.12b), and which subsequently recovered. Although other rocks were subject to the same procedures, only one other, LdCh, was of similar low strength. So although work by New (1976) and New and West (1980) shows that less stress is necessary to produce acoustic closure in stronger rocks, they would nevertheless be more resistant to compression of void spaces. Ultrasonic velocity data were not possible for LdCh following freeze-thaw and salt weathering but it is notable that this rock too experienced a reduction in porosity, a factor which could be explained in this case, by non-recoverable pore compression.

For SpaL, CalS and MetS no clear trend between specimens can be identified and there was considerable temporal variation in E_{dyn} values. Although some individual specimens showed an increase in E_{dyn} which is difficult to explain, many also showed a reduction which could reflect deterioration.

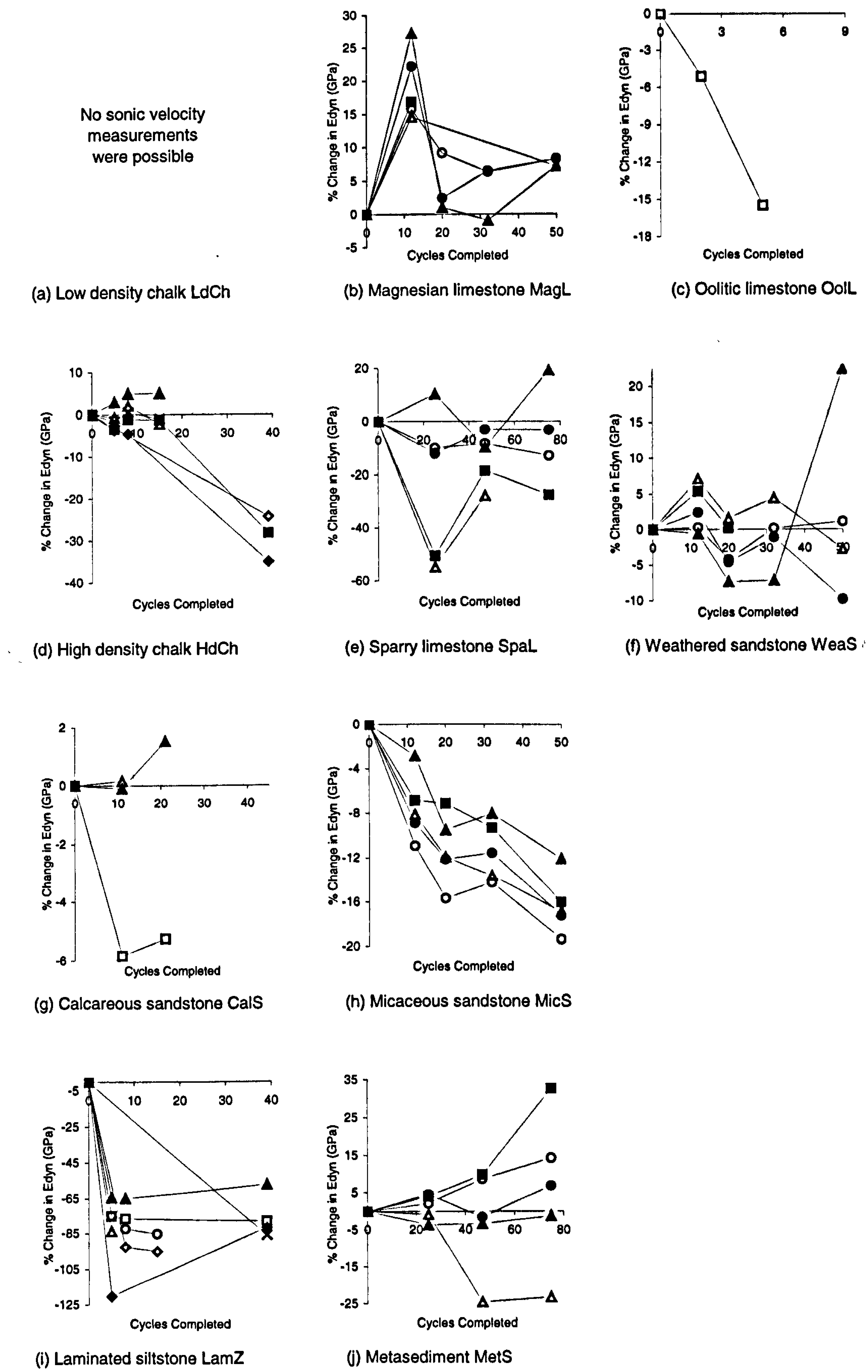


Figure 5.28 Percentage change in dynamic elasticity due to freeze-thaw

5.5.6.2 Salt weathering test

Converse to the freeze-thaw test, most samples (MagL, OolL, SpaL, WeaS, CalS, MicS) showed an increase in E_{dyn} due to salt weathering. This can be explained by the infilling of pores by salt deposits leading to a reduction in wave attenuation. These samples generally showed good accord between individual specimens. SpaL and MicS also showed an increase in E_{dyn} but there was greater fluctuation in the temporal trend and more variability between specimens. Comparison of these results with percentage change in n_e (Figure 5.13) shows that trends are as expected for most rocks, in that an increase in E_{dyn} corresponds with a reduction in n_e (MagL, OolL, SpaL, MicS). However, this is not the case for WeaS and CalS for which significant increases in n_e were recorded (Figure 5.13f and g). This suggests that some pores were filled with salt, as indicated by the increase in E_{dyn} and also by the pore size distribution data, while for others there was improved connectivity. New void space might also have been generated but if the total amount of unfilled void remained constant before and after testing, this would have resulted in an increase in both n_e and E_{dyn}. The reverse situation was found for HdCh, where despite an overall reduction in E_{dyn}, there was also a reduction in n_e (Figure 5.13d). In this case it is likely that the intense (strong) incipient fracturing which occurred in this rock was responsible for the reduction in E_{dyn}. The reduction in n_e suggests that pore infilling occurred and that the incipient fractures did not permit ingress of water though they clearly contributed to the overall void volume of the rock. Four out of five MetS specimens indicated negligible change in E_{dyn}, but one anomalous specimen showed a significant reduction. This particular specimen developed a persistent axial crack after one cycle of weathering, which progressively weakened and widened. It was not possible to measure E_{dyn} for LdCh and LamZ.

5.5.6.3 Wetting and drying test

With the exception of LamZ which showed a very significant decrease in E_{dyn} due to wetting and drying, the samples all showed a very small overall reduction, though there was commonly a moderate amount of intra-sample variation. This corresponds well with the actual pattern of deterioration observed. It does not correspond well with observations of percentage change in n_e, however, especially in the case of HdCh where change in n_e was comparable to that for LamZ. This is strong evidence that the apparent increase in n_e in HdCh represents increased connectivity to the surface and hence improved saturation, rather than any change in total volume of void. Had the latter occurred E_{dyn} would also have been expected to change more significantly.

5.6 Concluding Remarks

The overall aim of this part of the research was to investigate the fundamental mechanisms of rock breakdown with special reference to the role of existing flaws and other rock properties. The purpose was for an improved understanding of fundamental rock weathering processes and controls to be applied to deterioration of rockslopes at the mass scale. A range of findings have been discussed, including the relative roles of rock and environmental control, the importance of void-dependent and mechanical rock properties, the mode of deterioration and the role of pre-existing flaws. The implications of these results are discussed in section 6.21.

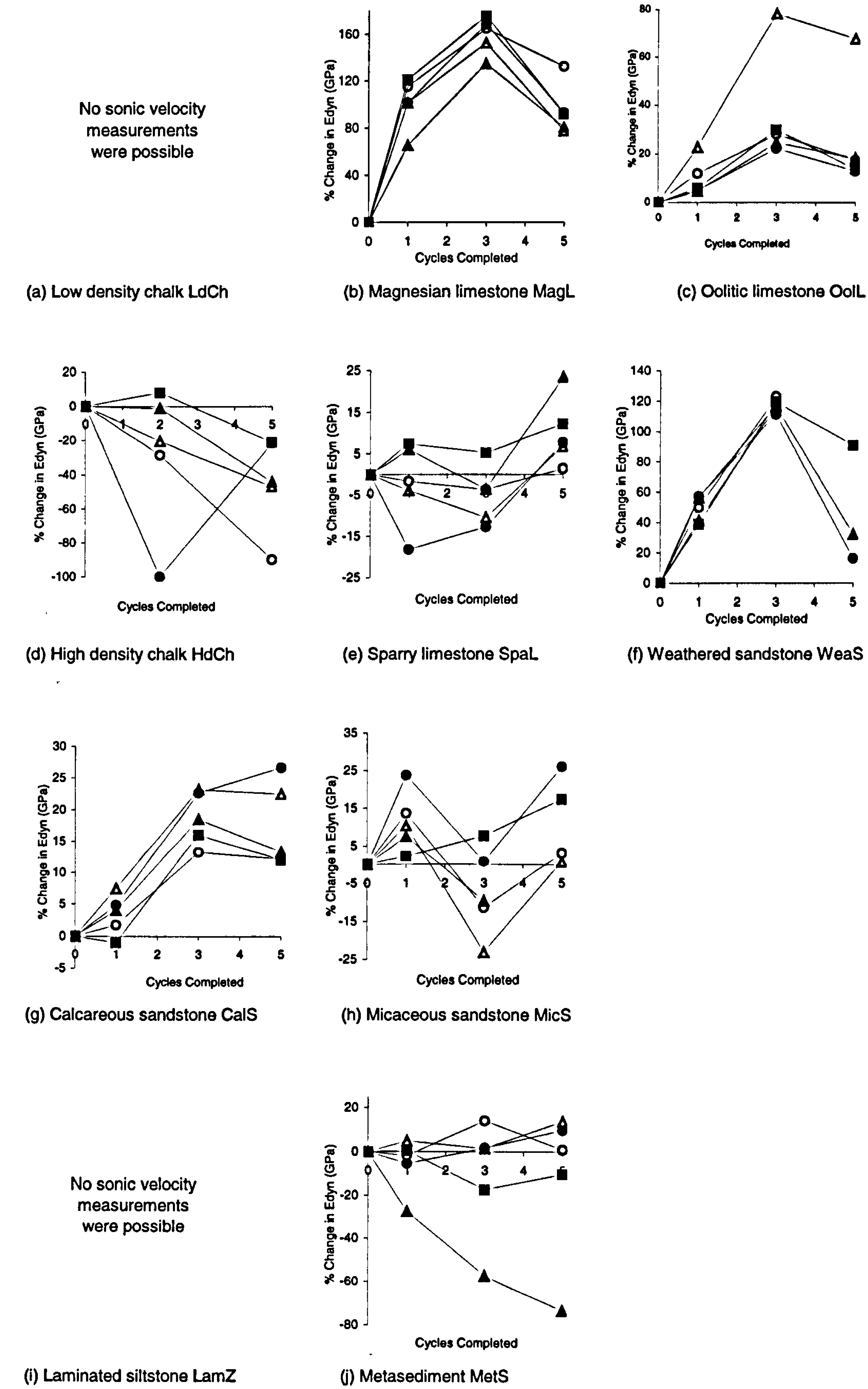


Figure 5.29 Percentage change in dynamic elasticity due to salt weathering

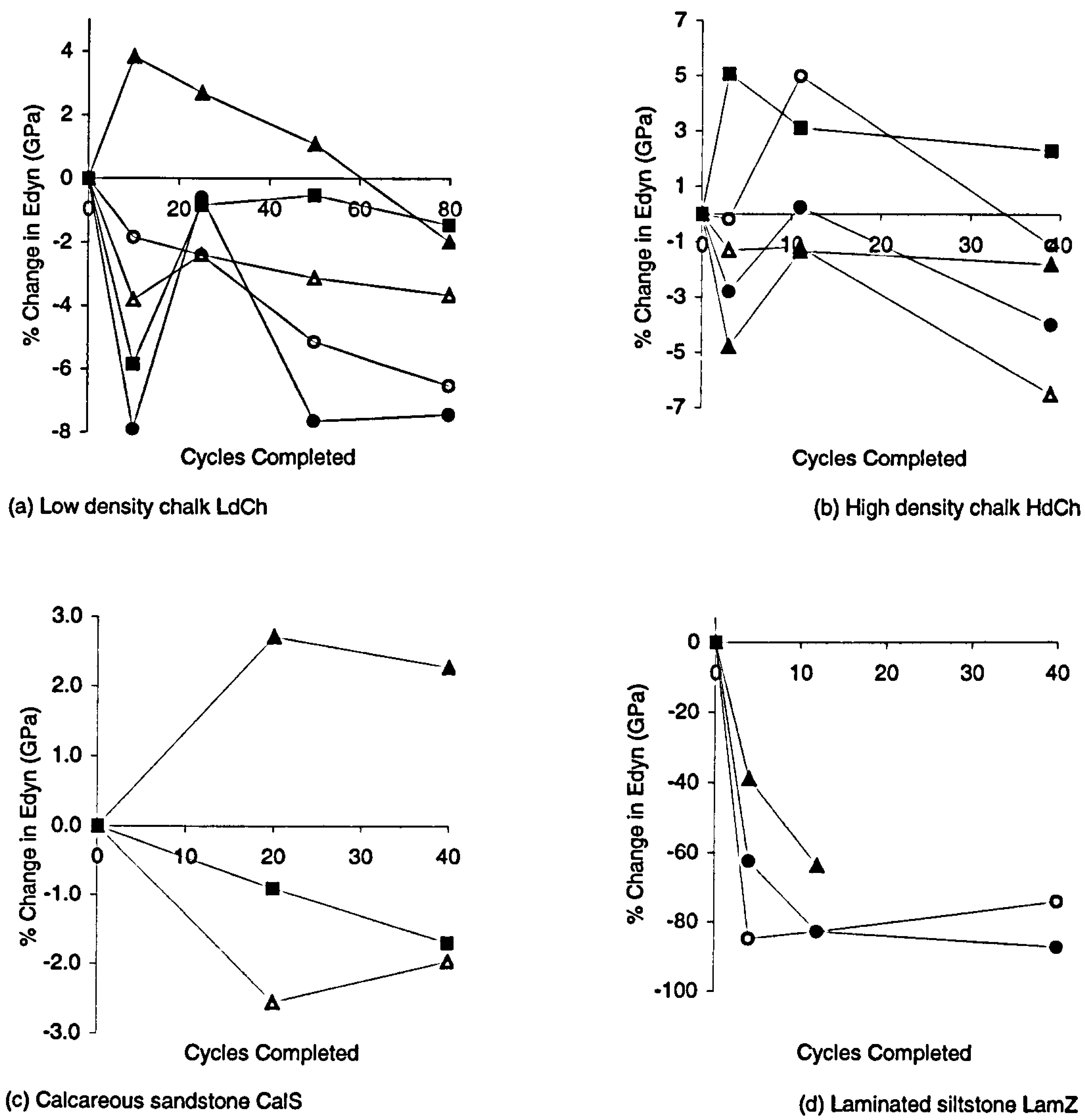


Figure 5.30 Percentage change in dynamic elasticity due to wetting and drying

PART TWO
ROCKSLOPE DETERIORATION
THE MASS SCALE

CHAPTER SIX
FACTORS AFFECTING ROCKSLOPE DETERIORATION

6.1 Introduction

In the second part of the thesis the emphasis changes to consider rockslope deterioration at the mass scale. In this chapter, the aim is to review the intrinsic and external factors influencing and controlling deterioration of excavated rockslopes (Figure 2.1) with reference to published literature.

6.2 Intrinsic Rock Properties Affecting Slope Deterioration

A good indication of the intrinsic properties considered to affect rockslope deterioration can be gained from a review of published data proformas and checklists (eg Geological Society Engineering Group Working Party 1977; Matheson 1983a; Nathanail 1992; Priest 1993b). Further indications can be obtained from rock mass classifications and slope hazard assessment schemes (eg Bieniawski 1989; Hack and Price 1993; Price 1993; Romana 1993; McMillan and Matheson 1997, 1998). However, caution is required with this approach since none of these methods were devised specifically to address rockslope deterioration but for a variety of other purposes. Decisions concerning which intrinsic properties of rockslopes to observe and record in the field were therefore, largely based on a review of published literature. The experimental work presented in Chapters Four and Five also indicated the most useful material properties to observe and record.

6.2.1 Rock material properties

The findings of the experimental rock weathering programme presented in Chapters Four and Five have several implications for the assessment of rock deterioration at the mass scale. These can be considered in terms of (i) the relative influence of environmental conditions on rock susceptibility to weathering; (ii) the role of material properties in rock breakdown; and (iii) breakdown mechanisms.

6.2.1.1 Influence of environmental conditions on rock susceptibility to weathering

It is clear for some rocks, notably chalk and oolitic limestone, that different *weathering processes* produce contrasting deterioration. It is reasonable to extrapolate from this the likelihood that these same rocks would also show considerable variation in deterioration with changes in *environmental conditions*. Consideration of environmental conditions in a more general way is helpful for field investigations where it is extremely difficult to identify specific weathering processes acting without detailed monitoring and investigation. This finding means that for some rocks, a small change in environmental conditions could result in substantial changes in deterioration susceptibility. On a large rockslope, for example, where there were changes in groundwater conditions, vegetation cover and aspect, this could produce contrasting deterioration zones within the same material.

The results also showed that occasionally, variation in weathering processes did not produce any modification in deterioration severity, but in the *mode* of deterioration. If the factors which control mode of deterioration operate in the same way at all scales, then this also has implications for assessment of deterioration for excavated rockslopes.

6.2.1.2 Role of rock properties

The experimental weathering programme showed that deterioration of most rocks, in fact, is much more controlled by rock type and material properties than by weathering processes. Analysis of correlation coefficients between rock properties and deterioration showed that no single property could be identified which controlled deterioration. Nevertheless, increasing susceptibility to weathering correlated reasonably well with three void-dependent properties: total connected pore volume, saturation coefficient and microporosity.

Attempts to relate rock mechanical properties with deterioration susceptibility were only partially successful. High strength rocks and rocks with high elasticity were clearly very durable, but for weaker rocks, no clear pattern was discernible. This means that a field estimate of rock strength alone is likely to prove a poor indicator of deterioration behaviour. Indeed there were several anomalous responses to weathering in the experimental study where weak rocks showed unusual resistance to breakdown. The exception to high strength rocks being more durable concerned those which contained major structural weakness. This is a good indication that rock flaws at all scales should be included in any assessment of deterioration susceptibility. Rock flaws, their type, persistence and penetration can be easily estimated in the field. Rock flaws are a major influence on both deterioration mode and severity, more so for stronger rocks. Linear weaknesses in particular, are more important than other types because they occur more commonly, they are more likely to lead to deterioration than other flaws, and are more likely to be densely spaced (eg closely spaced laminations).

For weaker rocks, however, other rock properties may be more important in controlling deterioration susceptibility than material weaknesses. For example, granular rocks are more susceptible to processes which exploit intergranular microcracks; an absence of clay minerals renders rocks more resistant to processes which involve hydration; extremely weak rocks are more likely to be severely affected by any weathering process though the mode of deterioration may not be the same in each case; and weak rocks with an interlocking texture may prove more resilient to deterioration than expected. It can be seen, therefore, that there are a wide range of structural, mechanical, textural, mineralogical and void-dependent properties which affect the mode and severity of rock deterioration. Of critical importance, is determining which property will have the dominant influence on deterioration response.

With considerable caution, bearing in mind the relatively small sample of materials used here, some general 'rules' can be applied:

- For strong rocks, the presence or absence of weaknesses (eg flaws) will dominate deterioration response;

- Medium to coarse granular texture will determine mode of deterioration for moderately strong rocks;
- Extremely weak rocks are likely to suffer severe deterioration regardless of the processes involved;
- Rocks with interlocking texture are likely to be more resistant than similar strength rocks with a non-interlocking texture;
- Some rocks may have complex pore structures or other characteristics which make it difficult for a reliable estimation of likely behaviour to be made. The deterioration response of these rocks may be more closely related to variations in environmental conditions;
- All other things being equal, rocks with higher pore volume, saturation coefficient and microporosity are likely to be more susceptible to deterioration.

Rock properties such as mechanical strength, structure and texture constitute clear definition of rock type as would be obtained from an engineering description, for example, a moderately weak, thickly laminated, moderately weathered, micaceous SANDSTONE (order after Hawkins 1986). This alone would provide a basis upon which to begin to estimate likely susceptibility to weathering.

6.2.1.3 Breakdown mechanisms

In section 5.5.4.4 a concept was proposed in which progressive internal modification of rock at the microscale would lead to damage at the macroscale. This has important implications for investigation of rockslopes. Deterioration in the form of material weakening might not be visible, but could occur even in the absence of any obvious indicator such as rupture. Further evidence to support this was shown by the reduction in point load strength experienced by many rocks after freeze-thaw. An absence of any manifestation of deterioration, therefore, cannot be taken as evidence that breakdown will not occur at a later date. Furthermore, this weakening renders the rock less able to withstand load, less able to tolerate further internal pressures due to weathering, and therefore more susceptible to deterioration.

In addition to understanding weathering susceptibility of rock, it is useful in field descriptions to include a description of the *current state of material weathering.* As well as contributing to overall assessment of intact rock strength, texture and permeability, for instance, it may also indicate susceptibility to deformation (Selby 1993). However, care is needed in the interpretation of weathering grade to ensure that it is relevant to *current* exposure conditions and not palaeo-environmental conditions. For instance, it is common to find palaeo-solution features exposed in limestone terrain. Further caution is required since current weathering grade may not take account of any external factors (eg stress or environmental conditions) which have recently been changed. For example, rapid degeneration of mudstones upon exposure is not uncommon.

6.2.2 Rock mass properties

The role of microcracks and pre-existing flaws in rock deterioration has been discussed in Part One of this thesis with respect to the material scale. Of interest now, is the role which discontinuities play in determining rock mass structure and of influencing its deterioration behaviour.

The importance of discontinuities in controlling stability of rock masses is widely acknowledged (eg Duncan and Goodman 1968; Geological Society Engineering Group Working Party 1977; Hoek and Bray 1981; Hencher 1987; Aydan et al 1992; Bell 1992b), indeed some would argue that failure rarely occurs in intact rock masses unless the material is extremely weak (Richards 1992). Others (eg Hoek 1973; Fookes and Sweeney 1976) also recognise the importance of time dependent progressive weathering in this context. Nevertheless, most deep-seated failures in rock masses are controlled by discontinuities and therefore their characterisation is of primary importance in any investigation of rock mass behaviour. This is reflected in the emphasis placed on discontinuity properties in many rock mass classifications (Deere 1963; Barton et al 1974; Selby 1980; Bieniawski 1989; Hack and Price 1993; Price 1993; Romana 1993), slope hazard assessment schemes (Nathanail et al 1992; McMillan and Matheson 1997, 1998) and slope stability analytical techniques (Burman et al 1975; Phillips 1971; Matheson 1983a, 1988, 1991; Hencher 1987; Nash 1987; Walton 1988; Barton1989; Giani 1992).

There are several reasons why discontinuities and their properties are so important in determining rock mass behaviour: (i) movement can occur along discontinuities; (ii) their spacing and orientation determine block shape and size, which also have a major influence on the mode of failure; (iii) discontinuities have an overriding influence on the rock mass structure and the spatial distribution of variations within it; and (iv) weathering processes exploit discontinuities, and some will *only* occur along them.

6.2.2.1 Movement along a discontinuity

Movement can occur along discontinuities. Whether or not movement can theoretically occur is best described in terms of factor of safety, which is the ratio of resisting forces to forces driving failure (or moments as appropriate, Bromhead 1996). The resisting forces relate to shear strength of the discontinuity plane, which can be determined from:

$$\tau = \sigma' \tan\phi' + c'A \qquad [6.1]$$

Where τ = shear strength; σ' = effective normal stress; ϕ' = angle of friction; and c' = apparent cohesion. A factor of safety equation for a slope is given below and illustrated in Figure 6.1:

$$F = \frac{c'A + (W\cos\alpha - U - V\sin\alpha) * \tan\phi'}{W\sin\alpha + V\cos\alpha} \qquad [6.2]$$

Where W is the weight of the slope overlying the discontinuity, A is the area of the block overlying the discontinuity, α is the dip of the discontinuity plane, U represents an uplift force due to water pressure and V represents cleft water pressure in a vertical crack at the rear of the overlying block.

The equation shows that the possibility of movement along a discontinuity is a function of several factors including the weight of the overlying block, pore water pressure (and cleft water pressure if appropriate), the frictional and cohesive properties of the discontinuity surfaces, and the area of contact between discontinuity walls. However, kinematic analysis also shows that for failure to occur, certain geometric requirements have to be satisfied. These concern the orientation of the discontinuity and the slope plane, and the angle of friction of the discontinuity. It is apparent, therefore, that a range of discontinuity properties contribute to the overall influence of discontinuities on rock mass behaviour. These include discontinuity orientation, spacing, persistence, type, wall strength and roughness, aperture, infilling, seepage, block size and the number of discontinuity sets. Definitions for these and guidance on their measurement, survey and recording techniques are given in several publications including ISRM (1978b), the Geological Society Engineering Group Working Party (1977), and Ewan et al (1981). Numerous attempts have been made to analyse statistical distributions of these properties and their inter-relationships (eg Pahl 1981; Einstein et al 1983; Dershowitz and Einstein 1988; La Pointe 1988; Kulatilake et al 1990; Sen 1990a, 1990b; Villaescusa and Brown 1990; Sen and Eissa 1992; Priest 1993a, 1993b), with particular emphasis on distributions of discontinuity spacing (eg Hudson and Priest 1979; Ladeira and Price 1981; Priest and Hudson 1976, 1981; Sen 1984; Sen and Kazi 1984). However, in the context of this thesis, it is critical to appreciate that all of this work relates to observations and measurements of major sets of discontinuities such as joints and bedding planes, and major isolated discontinuities such as faults. It does not generally relate to isolated, minor, surficial fractures of the type which would be commonly produced by weathering, blast damage and even localised stress release.

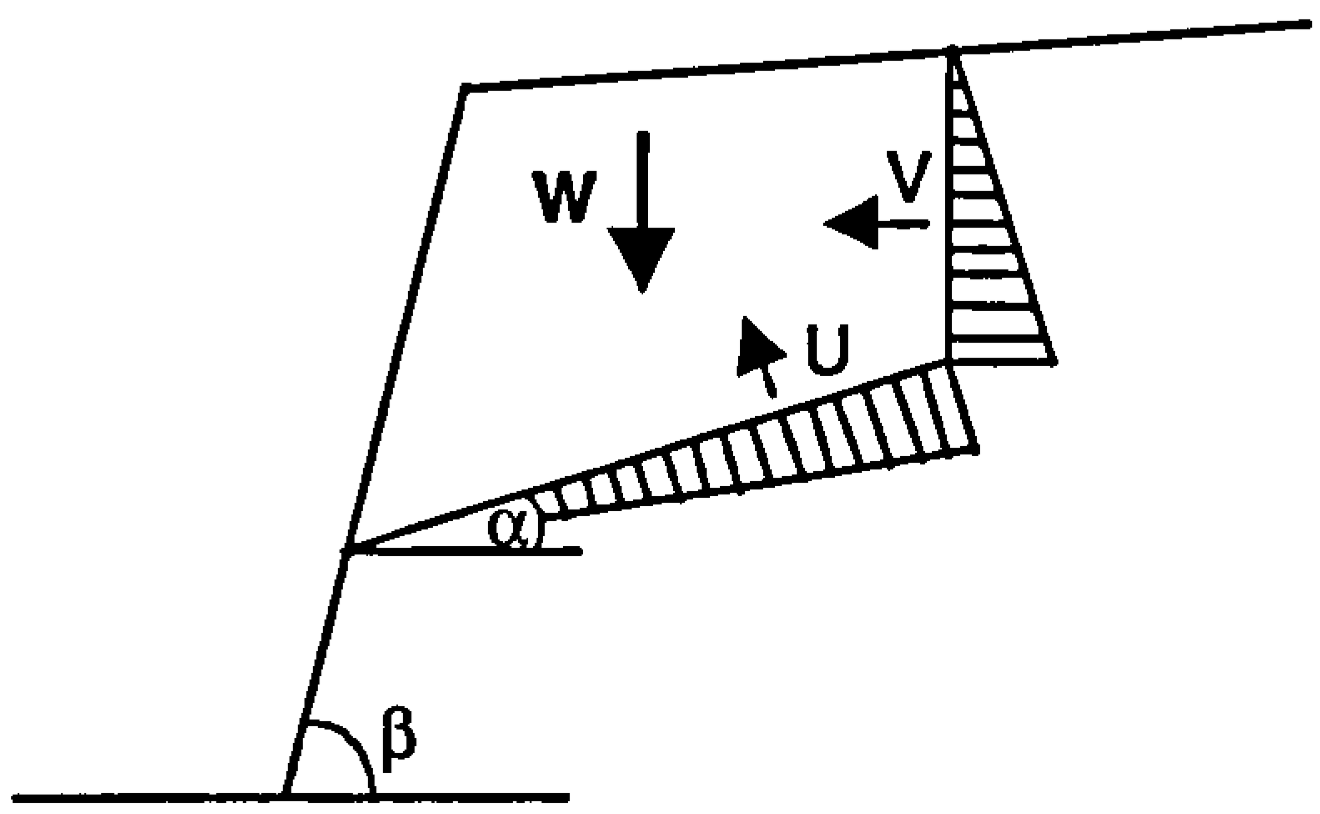


Figure 6.1 Diagrammatic representation of principal forces acting upon a slope

6.2.2.2 Mode of failure

Landslides have been defined as "a movement of a mass of rock, earth or debris down a slope" (Cruden 1991), while others prefer the term *mass movement* (Brunsden 1979b). Thus all types of downslope movement of rock along a discontinuity can be described collectively as landslides. Three main types of discontinuity-controlled failure are recognised for simplified 'static' analytical purposes (Bromhead 1996). *Planar* failures involve translational sliding of a block along a single discontinuity surface which daylights into the exposed slope face (Richards 1992). *Wedge* failures also involve sliding, but in this case along two intersecting discontinuity surfaces, the line of intersection of which daylights into the slope (Richards 1992). *Toppling* failure involves overturning of blocks about a pivot and occurs in steeply jointed rock masses with a steep slope face. Elements of both rotation and sliding can be involved in this mechanism (Figure 6.2), the proportion of each largely being a function of block shape (Hoek and Bray 1981). For heavily fractured or soil-like rock masses a fourth failure mechanism, *circular* failure is also recognised (Hoek 1973), where the material is essentially structureless. These principal failure modes were illustrated in Chapter Two (Figure 2.2).

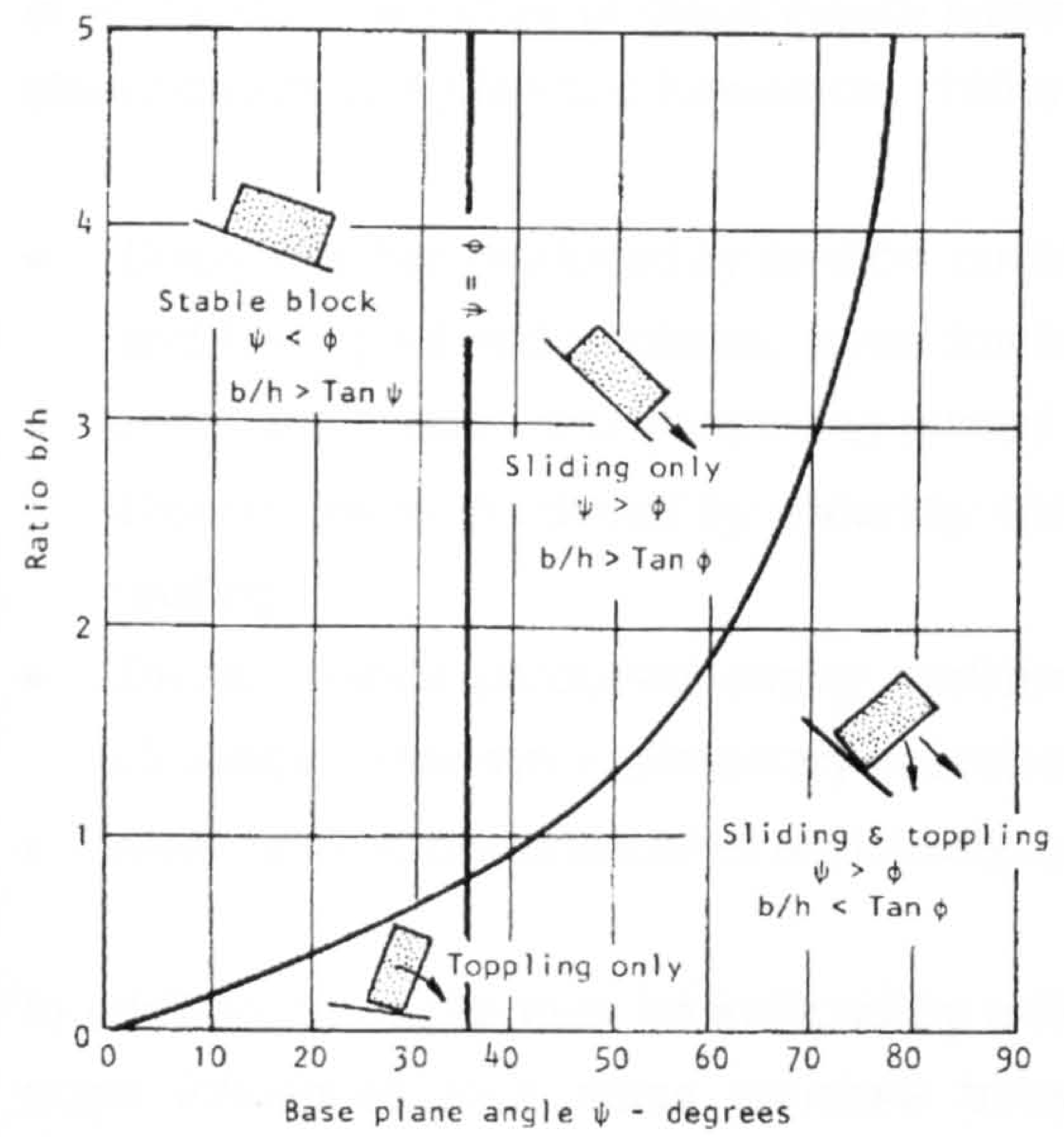


Figure 6.2 Conditions for sliding and rotation in toppling failures (*after Hoek and Bray 1981*)

Further types of failure are recognised by landslide practitioners such as rock creep, rock avalanche and rockfall (eg Dikau et al 1996). These tend to be analysed by magnitude-frequency and post-movement investigations rather than the 'static' methods used by engineers. Rock creep, also known as rock flow, is a very large, deep-seated, slow deformation of a rock mass, often occurring along shear planes (Dikau et al 1996). Rockfall tends to occur in highly fractured rock masses (Carson and Kirkby 1972) but major joints may define a failure surface, often forming a wedge-like hollow (Dikau et al 1996). Walton (1988) points out that rockfalls also occur commonly in rock masses with notable spatial variations in material properties. These conditions are similar for the generation of rock avalanches except that these usually involve failure of a rock mass on a much larger scale. The defining feature of a rock avalanche, however, is its post-failure behaviour, which involves movement of the debris over great distances and at high velocity (Dikau et al 1996). Unlike rockfalls, where the principle movement is fall (though some initial sliding may occur), the later movement of rock avalanches often involves flow. Carson and Kirkby (1972) have proposed a simple classification of rockslope failure modes which are weathering-induced (Figure 6.3). These comprise (a) slab-failure, (b) rock avalanche, (c) rockfall and (d) granular disintegration. This is a slight departure from conventional classifications because the latter mode does not depend upon the presence of discontinuities at the rock mass scale. It is important to remember that while macro discontinuities are critically important to rock mass behaviour, they do not control all forms of slope failure where time-dependent weathering processes operate at the material scale.

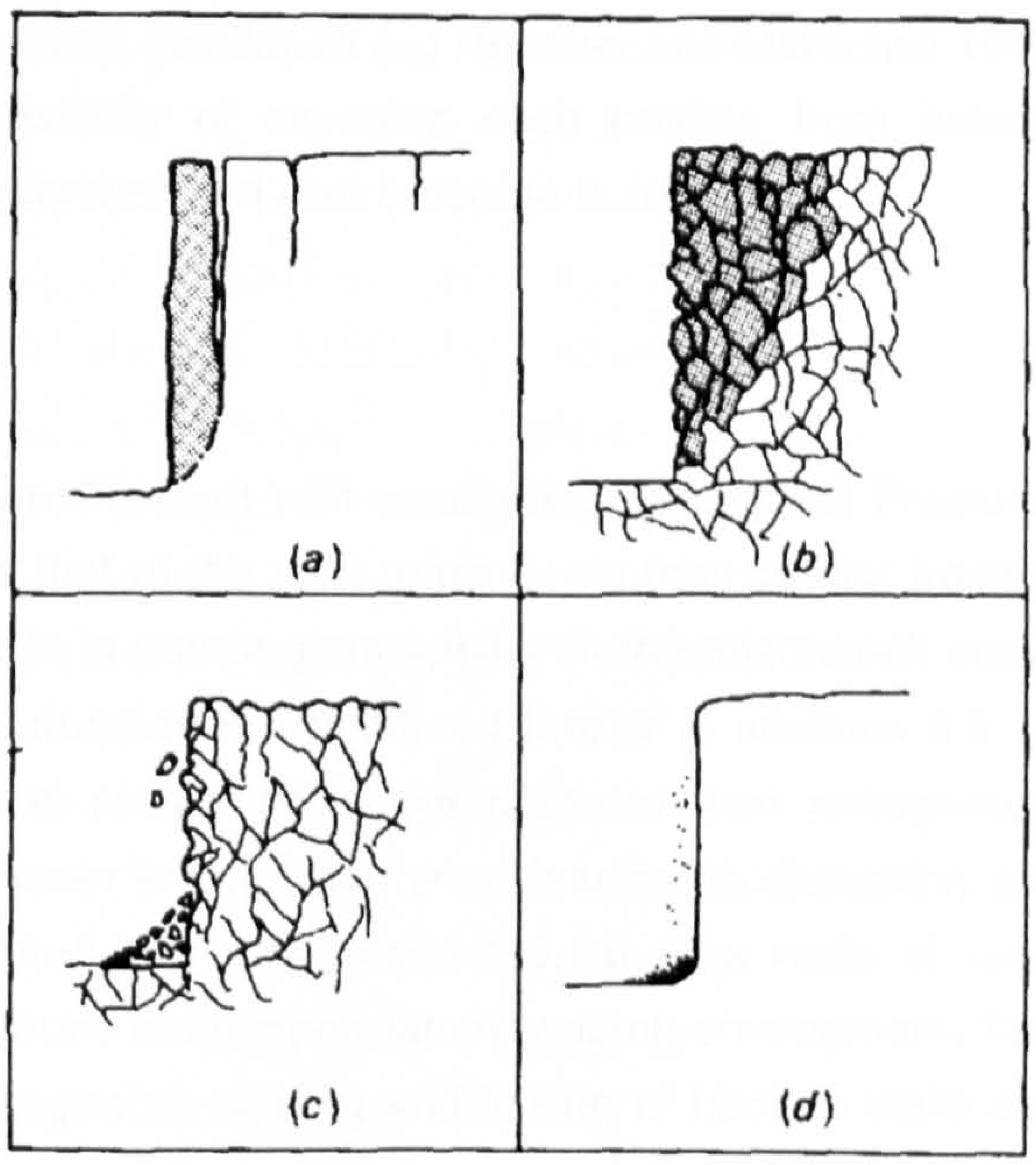


Figure 6.3 Classification of weathering-related rockslope instability (*after Carson and Kirkby 1972).*

6.2.2.3 Rock mass structure

The structure of a rock mass, the spatial distribution of variations within it and its geotechnical properties are largely controlled by the type and genesis of discontinuities present (Geological Society Engineering Group Working Party 1977; Rawnsley 1990; Rawnsley et al 1990). The range of discontinuity types recognised are listed below, broadly based on a classification by Aydan and Kawamoto (1990):

- *Discontinuities produced by tension:* cooling joints, tectonic joints, rebound fractures, faulting and folding-related fractures, some fractures related to igneous intrusions, desiccation and shrinkage cracks, and weathering-related fractures.
- *Discontinuities produced by shearing:* some joints related to igneous intrusions, folding and faulting.
- *Discontinuities produced during sedimentation:* bedding planes and laminations, shaley cleavage, other syn-sedimentary structures.
- *Metamorphic discontinuities:* schistocity, gneissose banding, foliation, slatey cleavage.

In addition, fractures may be induced by anthropogenic activity such as blasting. Discontinuity origin influences rock mass structure because it can determine surface morphology and aperture of a fracture, which in turn, affect roughness and therefore frictional properties (Aydan and Kawamoto 1990). Different types of discontinuity also have characteristic spacing and persistence, and in some cases orientation (eg Hencher 1987). Tectonic joints, for example, tend to have a regular spacing and often occur as several sets of intersecting fractures. This can create an orthogonal blocky structure. Alternately, the presence of regular cooling joints often produces rock masses with a characteristic prismatic or columnar structure. Well developed slatey or shaley cleavage usually results in a rock mass which is fissile. Fractures related to folded beds and to shear zones tend to produce local intensely fractured zones, with rock mass properties distinct from the adjacent mass. Other variations in rock mass properties can be brought about in sedimentary sequences with significant changes of lithology (eg interbedded sandstones and shales). Time dependent processes such as weathering, stress relief and changes in water flow may also bring about spatial variations in rock mass properties. Deeply weathered granite slopes in which corestones have developed (eg Hencher and McNicholl 1995) are an excellent example of this. The possibility of exposing such profiles from palaeo-weathering rather from than current day conditions should also be borne in mind.

6.2.2.4 Weathering along fractures

There are a number of ways in which fractures interact with weathering processes: Fractures provide flow pathways for water. This means that at the rock mass scale they control fracture permeability while also having an important role in porous permeability at the microcrack scale. The role of ground and surface water in deterioration is considered further in sections 6.3.1.2 and 6.3.1.3. Blocks which become trapped in cracks might contribute to their enlargement (section 6.3.1.4). Vegetation may exploit or even enlarge fractures, leading to disruption and also chemical effects. This effect is considered more in section 6.3.1.6. The walls of open fractures may be preferentially weathered because of their permanently damp environment. This can induce material decomposition and disintegration, scaling and flaking of fracture walls and the accumulation of detrital material in cracks.

6.3 External Factors Affecting Rockslope Deterioration

6.3.1 Environmental conditions

The influence of environmental conditions on rockslope deterioration is largely related to climatic controls, particularly moisture and temperature regime. These, together with associated controls such as cloudiness, wind speed, aspect and vegetation cover are considered now.

6.3.1.1 British climate and weathering

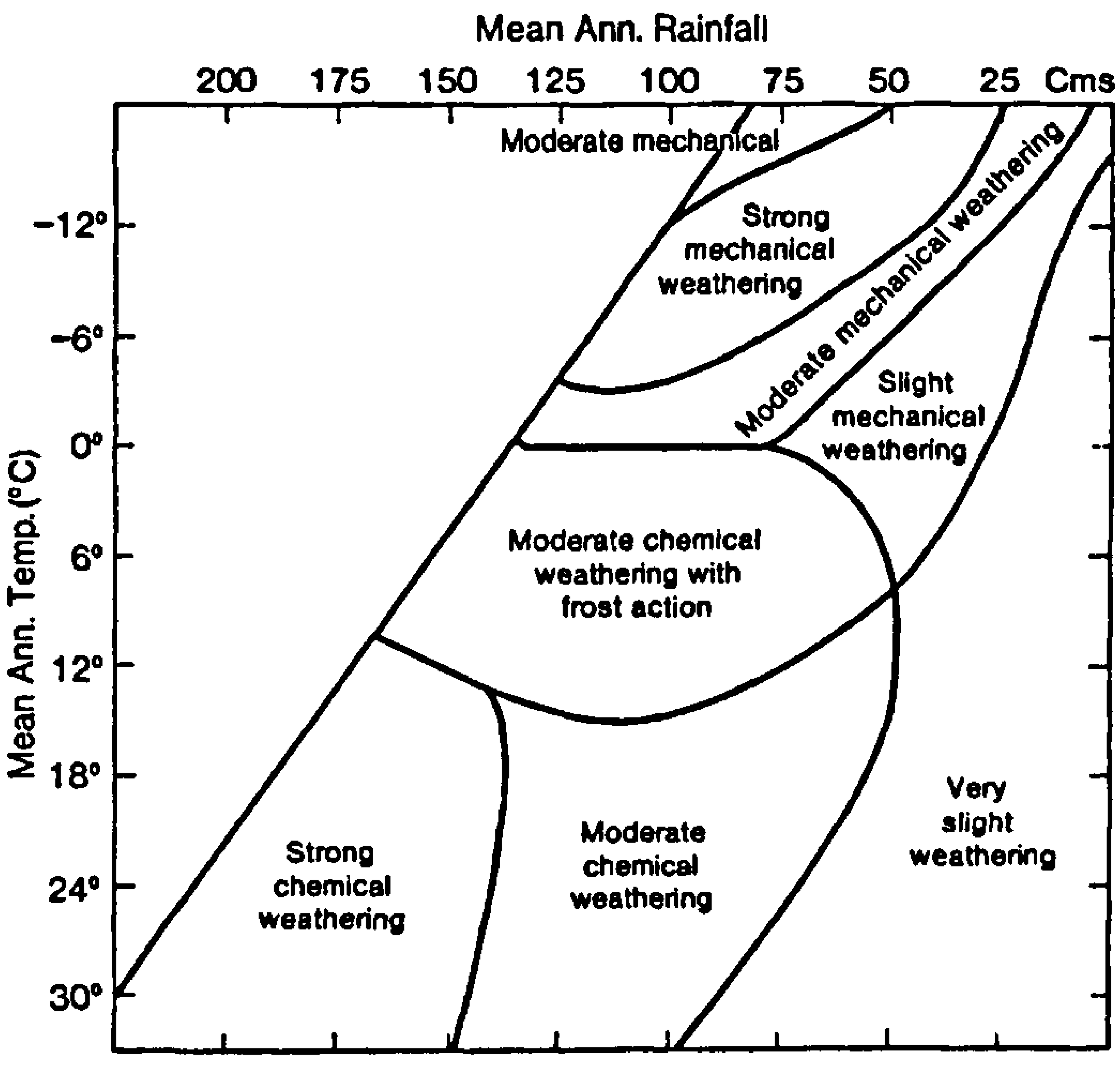


Figure 6.4 Categories of weathering in relation to temperature and precipitation (*after Peltier 1950*)

Several attempts have been made to classify climatic regions of the world (eg Köppen, in Barry and Chorley 1982) and to link them with likely weathering type and intensity (eg Peltier 1950, given in Figure 6.4; Strakhov 1967). Britain lies in a maritime temperate zone and according to the classification of Peltier, experiences 'moderate' chemical weathering intensity and 'weak' frost weathering intensity. Peltier's (1950) classification is based on the assumption that chemical weathering thrives in warm, moist environments. However, recent studies have suggested that chemical weathering can in fact occur in much cooler, arid climates (Douglas et al 1991; Whalley et al 1982). The classification also assumes that frost is the primary agent of weathering in cooler climates. However, the precise relationship between frost shattering efficacy, freeze-thaw cycles, freezing intensity and duration are unclear. Furthermore, an increasing number of workers are beginning to doubt that the importance attributed to frost weathering in cooler climates is justified and to suggest a variety of other mechanisms that may be responsible for rock fracture (eg Whalley et al 1982; Hall 2000; Whalley et al 2000).

The simplification inherent in Peltier's model also masks huge regional and local variations in climate which may occur due to the differences in the range of air masses affecting Britain (Barry and Chorley 1982) as well as topographic influences. For example, mean annual rainfall on the west coast of Britain is typically 1140mm, but this increases to a staggering 3800mm for western mountain areas such as the Lake District, Snowdonia and the western highlands of Scotland (Barry and Chorley 1982).

In the geological past, Britain has been subject to climatic regimes which contrast with those of the present day. Weathering features which may be exposed in a new rockslope, therefore, may be the result of palaeoclimatic weathering conditions and should be interpreted in that light.

6.3.1.2 Surface water

Precipitation is the primary input to surface runoff and infiltration. In very weak rock and soil-like materials, precipitation can also lead directly to surface compaction and erosion by raindrop impact, particularly in regions where heavy storms are common (McIntyre 1958). A component of surface runoff may also be supplied from groundwater seepage but the spatial and temporal distribution of such seepage is difficult to predict without detailed investigation. Surface runoff is usually channelled. In highly weathered or soil-like rock material turbulent flow may lead to the development of rills, or gullying in severe cases, and substantial slope incision can result. On tougher rockslopes surface runoff is likely to be concentrated into pre-existing channel-like forms resulting from the rock structure (eg chutes, open fractures and exposed bedding planes) or as a bi-product of excavation (eg sloping benches and drainage channels). Occasionally, surface runoff may occur as overland flow, independent of any channel forms.

The erosive capacity of surface runoff is partly a function of its flow velocity, which in turn, increases with increasing slope gradient and slope length. The erodibility of the slope material, however, is also strongly influenced by the shape and roughness of the slope (Gerrard 1981). While surface sheetwash is more likely on uniform slopes, channelled flow on irregular surfaces can be more damaging locally.

6.3.1.3 Groundwater

Groundwater, whether present in the intergranular pores of a rock or within discontinuities, has the potential to influence deterioration of rockslopes in a number of ways. This is indicated by the frequency with which rockslope failures occur in association with high pore or cleft water pressures (eg Kiersch 1964; Schumm and Chorley 1966; Suwa et al 1983; Pomeroy 1984; Yamaguchi and Shimotani 1986; Gagen 1988; Clark et al 1993).

Reduction of effective stress: The presence of water in the matrix of a rock reduces material strength, an effect which is particularly notable in more porous rocks (Waltham 1994). This is due to the effect of pore water pressure which acts against the confining stress. A similar effect occurs due to cleft water pressures (ie water in fractures). The effect of a reduction in strength is to render the rock material and mass more susceptible to the processes of deterioration.

Modification of discontinuities: Groundwater flow may cause the enlargement of discontinuities, whether they be at the rock mass scale or at the level of intergranular cracks. This can occur by detachment of material from fracture walls, dissolution, or the flushing out of infilling materials. The resulting increase in aperture causes a reduction in rock mass strength and modifies the frictional and cohesive properties of the discontinuity, particularly where infilling materials are removed. Water flow in discontinuities can also result in the *deposition* of infilling materials.

Weathering: The presence of moisture in intergranular pores of rock or within discontinuities promotes physical and chemical weathering. The release of groundwater as surface seepage also has implications for both chemical and physical weathering: (i) Chemical agents may be transported to the external slope environment, where new weathering reactions may take place, or rates of weathering may be enhanced. (ii) Enhanced physical weathering (eg abrasion and erosion) may occur at points of seepage onto the slope face.

6.3.1.4 Rock weathering and climatic conditions

The presence of moisture, together with temporal fluctuations of moisture and temperature, have a direct role in most physical and chemical weathering processes. Many of these processes, which operate at the material scale, were described in Chapter Two, but some mechanisms such as block wedging, operate at the mass scale and were not discussed earlier. Block wedging occurs when a fragment of rock which drops into an open crack during the expansive phase of thermal or hydrologic cycles exerts considerable force on the walls of that crack during the contraction phase. While there is good anecdotal evidence of the occurrence of this process it has not been investigated experimentally and its mechanisms are unclear. Observations suggest that locally, the process makes a significant contribution to fracture dilation and extension.

In addition to the more obvious moisture and temperature influences on rock weathering, there are other less direct climatic influences. For instance, cloudiness influences temperatures at the Earth's surface, so that the greater the cloud cover, the less likelihood of a frost occurring. The duration of snow cover is also significant because although it can reduce the severity of freezing but increases freezing duration. Snow cover also reduces the number of freeze-thaw cycles. Snow cover also protects the rock surface from the action of other weathering and erosive agents (Ollier 1984). Higher wind speeds increase moisture evaporation, leading to quicker drying out of a rock surface, and may increase rain penetration of rock (Henriques 1993). Wind speed may also affect temperature. Slopes may be subject to additional climatic extremes if particularly exposed, at high altitude, situated in frost pockets or sites of cold air drainage, or subject to rapid insolation or drought.

The extremes of climate may intensify 'normal' weathering effects and therefore deserve special consideration. For example, prolonged drought is likely to cause removal of moisture from a much greater depth in the rock than is usually the case. Exceptionally high intensity rainfall could also lead to rainsplash erosion in a weathered rock which would normally resist damage. Other climatic factors such as relative humidity may also be important locally, especially where salt weathering is active.

6.3.1.5 Slope aspect

The orientation of a rockslope, known as its 'aspect', can greatly influence deterioration for two reasons. One is that slope orientation determines intersection of discontinuities with the slope plane, and thus determines the structure of the rock mass at the surface, and the other is that it influences the range of climatic effects upon it. The rock mass structure can provide a significantly greater control on slope behaviour than climatic effects.

Aspect may determine the hours of sunshine received by a slope, its exposure to prevailing winds and rainfall, mean temperature range, and the number and intensity of frosts. However, it has been difficult to reach a consensus as to the exact relationship between aspect and climatic conditions despite numerous studies, perhaps because the effects of other local factors are difficult to eliminate. Robinson and Williams (1998) found, in a study of the weathering of sandstone gravestones, that west facing stones were much more weathered than those facing east, although there were some anomalies. They suggested that since the prevailing wind direction was westerly, west facing sides would therefore more commonly be rain-soaked, whereas the east facing sides were generally more sheltered and remained dry for much longer. Exposure to wind and rain as a key factor in determining differential weathering rates was also indicated by Mottershead (1994, 1997) in work on coastal sandstone wall weathering. However, Robinson and Williams (1998) highlighted the difficulty in their work of reconciling the fact that west facing stones also receive sun in the hottest part of the day and should therefore dry out more quickly than east facing stones.

A further study of gravestones by the same authors working in the same area of southern England, but on a different sandstone, showed the opposite result, with greater weathering on east facing sides (Williams and Robinson 2000). This was supported by a similar study of the weathering of sandstone churches in the area (Robinson and Williams 1996). The former study also indicated greater weathering on some north-east facing stones than those facing south-west, suggesting that a northerly aspect predominates over a southerly aspect in terms of weathering intensity. These two studies appear to suggest that greater weathering intensity can be expected on sides facing away from prevailing wind and rain. In the study of gravestones (Williams and Robinson 2000), it was also noted that there was no change in weathering asymmetry even when stones were permanently cast in shade from evergreen trees, indicating that the role of temperature fluctuations was of little consequence. The present author suggests that while short term temperature fluctuations may not be distinctive for different aspects, total solar radiation input is critical: The key factor in determining weathering is likely to be long term retention of moisture. Such a condition would favour various chemical weathering processes as well as granular disintegration from frost or salt action. In the second tombstone study reported above (Williams and Robinson 2000), the east facing sides would have been damp for a much greater proportion of time than those facing west because while the latter were subject to prevailing wind and rain, exposure to solar radiation in the hottest part of the day would ensure their rapid drying out. Conversely, although situated in a relatively sheltered environment, east facing sides would nevertheless receive direct rain from time to time, but would not dry out very quickly. This would be particularly evident for those stones which were permanently shaded. East facing sides would also remain cooler.

The contrasting result of the first gravestone study (Robinson and Williams 1998) may indicate that a delicate balance exists whereby if solar input is insufficient to evaporate moisture from stones facing the prevailing wind and rain direction, then moderate weathering will result. The opposite may also be true for east facing sides. The likelihood is, that minor changes in exposure, altitude and even stone colour could result in changes to the radiation and moisture balance. Work by Meierding (2000) on tombstones in the United States has confirmed that atmospheric pollution has a major influence on stone weathering and it is, therefore, also possible that local variations in industrial activity could account for some of the anomalies described.

Ollier (1969) proposes the following model for the effect of aspect on slope weathering in the northern hemisphere: (i) In cold regions subject to frequent frosts, south-facing slopes are susceptible to greater thawing and more freeze-thaw cycles than north-facing slopes, and therefore greater mechanical weathering. (ii) In cool temperate regions subject to occasional frosts, south-facing slopes receive more sunshine than north-facing slopes and so vegetation cover establishes more easily, leading to enhanced biotic and chemical weathering. North-facing slopes are subject to limited mechanical weathering from occasional frosts. (iii) In warmer temperate regions, south-facing slopes may not develop full vegetation cover due to high midday temperatures and occasional drought conditions. Therefore, less biotic weathering occurs than in the previous example, but the total amount of erosion and removal of weathered products is greater. In a UK context, these three climatic types can be compared, respectively, with (i) the most northerly British latitudes and high, exposed mountain areas; (ii) typical inland and coastal locations in the Midlands and north of England and Wales; and (iii) particularly sheltered locations and the south of England and Wales.

Although simplifications such as these can be made and are useful for general planning purposes, assessment of the influence of aspect at a particular rockslope is best undertaken with reference to the fundamental principles involved. These are summarised below on the basis of established principles and interpretation of the published literature.

Northerly aspects do not receive any direct solar radiation and therefore moisture is likely to evaporate very slowly, so damp surfaces may be retained for long periods of time. Diurnal changes in temperature are less extreme than for other orientations, though mean minimum temperatures will be lower, and cool conditions can persist. This means that freeze-thaw and thermal cycling is less extreme than for other orientations. Vegetation will also be difficult to establish.

All other aspects receive direct solar radiation but the intensity of heating increases clockwise from east. So, east facing slopes receive insolation in the coolest part of the day, while west facing slopes receive the greatest solar input in the warmer part of the day and are more likely to be subject to drought conditions in warmer locations or seasons. West facing slopes are also likely to be subject to more frequent rain and wind since westerly air masses are the most common to affect Britain (Barry and Chorley 1982). West and south west facing slopes also dry out more quickly after rainfall and therefore experience more rapid wetting and drying than other aspects, as well as more frequent and more intense freeze-thaw and insolation cycles. Robinson and Williams (1996) suggest that this may lead to more rapid surface crust development or case hardening which would subsequently protect the rock from weathering. Under marginally cold conditions, north facing slopes will experience freezing more often than south facing slopes because they do not receive the same warming by day. However, when long periods of frost occur south west facing slopes experience most freeze-thaw cycles because they are preferentially warmed by solar radiation. East facing slopes remain cool for long periods of time and also retain moisture for longer. South west facing slopes are likely to provide the most favourable orientation for vegetation growth.

Many of the ideas on the influence of aspect on weathering pre-date recent modelling of frost weathering (Walder and Hallett 1985) where the relevance of the *frequency* of freeze-thaw cycles is questioned, and more emphasis placed on the role of freezing *duration and intensity.* If the 'water migration' theory of frost weathering (Walder and Hallett 1985, 1986) is correct, then the assumptions made about the role of freeze-thaw cycles particularly for northerly and southerly aspects may be ill-conceived. For instance, it is assumed that south facing slopes undergo more freeze-thaw cycles and are thus subject to greater mechanical weathering. If, in fact, freezing duration supersedes freeze-thaw cycles in importance in frost weathering, then northerly aspects would be subject to greatest damage by this mechanism. This is indicated in the work of Williams and Robinson (2000) as discussed above. The fragmentation of southerly slopes often attributed to freeze-thaw, might in fact, reflect both physical and chemical damage due to vegetation, or to a variety of other processes.

6.3.1.6 The influence of vegetation on rockslope deterioration

Vegetation can be regarded as having three potential effects on rockslope deterioration: It may be involved in a variety of bio-mechanical and bio-chemical material weathering effects; it may modify the weathering environment at and near the slope surface; and it may interact with the processes of slope erosion. Some of the ways in which vegetation interacts with slopes to affect instability are shown in Figure 6.5.

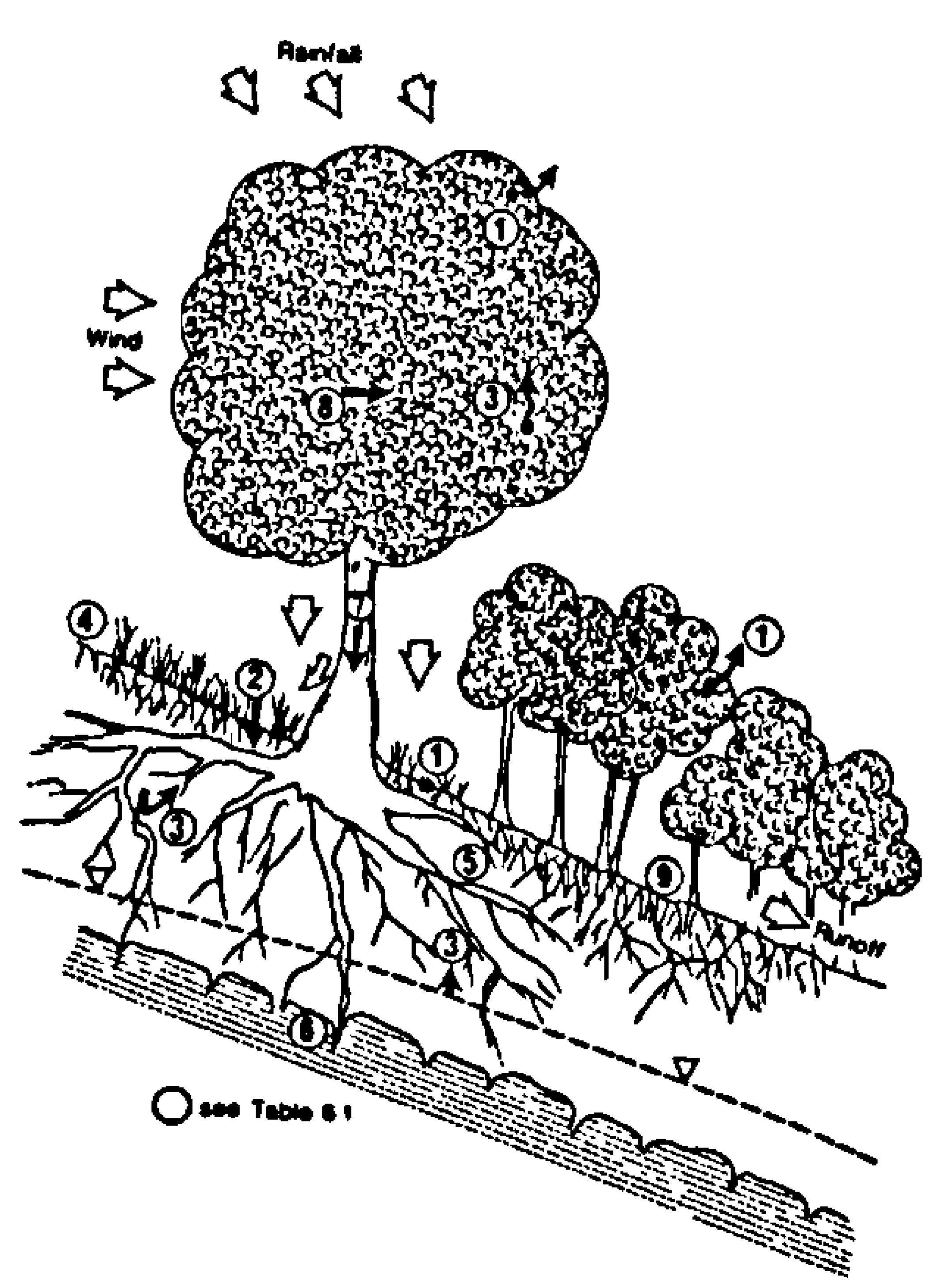


Figure 6.5 Idealised interactions between a slope and vegetation in ways which affect stability (*after Greenway 1987*).

Hydrological factors: 1 = foliage interception of rainfall; 2 = increased infiltration capacity; 3 = reduction of pore water pressure due to moisture extraction; 4 = desiccation cracking of the soil.

Mechanical factors: 5 = root reinforcement of soil; 6 = tree root anchorage of soil; 7 = surcharge due to weight of trees; 8 = windloading; 9 = binding of soil and rock particles at the surface.

(a) Biotic weathering

The wedging apart of rock by the growth of plant roots is well known anecdotally (eg Ollier 1984; Dubin et al 1986; Mitchell 1988; Coppin and Richards 1990; Selby 1993; Gellatley et al 1994;

Winkler 1994; Cherubini and Giasi 1997). However, there do not appear to be any data corroborating the mechanism involved. There are two ways in which the potential role of plant root growth in rock breakdown can be viewed: (i) Plant root growth is opportunistic and simply exploits existing cracks, or (ii) roots penetrate tightly closed cracks and their growth produces sufficient mechanical force to overcome the tensile strength of the rock causing fracture dilation and extension. The relative balance between these two possibilities remains unclear. However, axial pressures of 3MPa exerted by growing plants indicate that mechanical action alone would be insufficient to cause fracture in all but the weakest of rocks (Bland and Rolls 1998 - source of data not attributed). In either case, it is clear that bio-chemical reactions between plant roots and fracture walls has considerable potential to enhance rock breakdown.

There is growing evidence (eg Moses and Smith 1994; Viles and Pentecost 1994; Simms 2000) of the importance of bio-erosion in rock breakdown. Lower plants such as bacteria, algae, lichens, mosses and fungi are known to contribute to rock mass deterioration. For instance, lichens can live on bare rock surfaces and exploit cracks in rock, extracting nutrients by ion exchange (chelation), a process which is directly involved in the mechanical and chemical alteration of minerals. The release of carbon dioxide into accumulated soil due to respiration of plants and animals also enhances dissolution effects. Lower plants might cause granular breakdown due to root growth, and the action of burrowing organisms causes mixing and transfer of weathered materials. This in turn, may increase the surface area available for subsequent chemical attack (Brunsden 1979a). Organic matter accumulation from the decomposition of vegetative material may create an acidic environment at the rock surface, enhancing solution and other chemical effects, and also increasing soil moisture retention.

(b) Modification of the weathering environment

Plants modify the microclimate near the rock surface, particularly rock temperature, which can influence freeze-thaw and insolation processes. Plants increase shading, for instance, and may reduce wind velocity (Brunsden 1979a). The presence of vegetation also modifies the rock surface humidity environment and may therefore influence wetting and drying and salt weathering processes. Vegetation may also reduce evaporation of surface moisture, but this may be counteracted by the increase in moisture removal by transpiration.

(c) Vegetation and slope erosion

The presence of vegetation on a slope has several direct benefits for slope erosion and stability: (i) Vegetation may physically protect the rock surface from abrasion by wind, flowing water and freefall debris impact. It also intercepts precipitation directly, thereby limiting raindrop impact, and thus reduces the total volume of moisture reaching the slope surface. (ii) Vegetation adds to the 'roughness' of a slope and can obstruct surface flow, reducing its velocity. (iii) Plant roots have significant tensile and shear strengths (Greenway 1987; Luke 1988) and can therefore provide reinforcement in the upper part of a slope, significantly increasing factor of safety for shallow soil movements (Ingold 1988). This is akin to the action of a geosynthetic membrane (Cherubini and Giasi 1997). Norris and Greenwood (2000) have even developed a modified factor of safety equation for slope stability which incorporates a term for root reinforcement.

Vegetation may also modify the erodibility of surficial materials which can have both adverse and beneficial effects. For instance, the accumulation of organic material may enhance chemical weathering. Also, the rate of soil moisture removal is increased by transpiration and this may lead to more rapid drying out of the slope surface. This removal of moisture, on the one hand, could inhibit chemical and physical weathering and reduce pore water pressures (Ingold 1988) but on the other hand, could generate more frequent wetting and drying cycles.

In addition to the biotic weathering effects described above, adverse effects on slope erosion and stability from vegetation can also be recognised: (i) The roots of plants provide pathways for moisture infiltration into the rock material, so that although the amount of precipitation reaching the surface is reduced, the proportion of that which infiltrates is increased (Luke 1988). (ii) Higher plants, particularly large shrubs and trees, can de-stabilise slopes due to windthrow. (iii) The death of a woody plant, the roots of which have hitherto provided soil reinforcement, can lead to collapse of a substantial volume of material bound up in the root system.

6.3.2 Static stress conditions

In this section the effects of artificial excavation on the internal stress state of a rock mass are considered with particular emphasis on the formation of stress release fractures. Internal rock stresses may be derived form four principal sources: thermal stress, tectonic stress, gravitational or topography-related stress and residual stress (Selby et al 1988). When a rock mass is excavated and a new surface created, two major changes occur to this internal stress system: First, relaxation of confining load leads to rebound (Nichols and Collins 1991); and second, pre-existing rock stresses are re-distributed, concentrated or mobilised (Yatsu 1988). Different types of internal stress are considered below.

6.3.2.1 Types of internal rock stress

Tectonic stresses: Tectonic stresses arise due to the movement of lithospheric plates and on a large scale can set up compressive or tensile forces in rock masses at the surface. The presence of joint sets and faults is evidence of tectonic stresses, but these may relate to stresses which are no longer active, or may have substantially changed in direction or magnitude.

Thermal stress: Thermal stresses arise when temperature changes cause expansion or contraction in a rock mass which is confined. The cooling of lava at the surface, for instance, leads to contraction, which if resisted, will generate high tensile stresses. If these are greater than the tensile strength of the rock, cooling joints will form. On the same principle, cyclic expansion and contraction due to diurnal temperature fluctuations may also lead to rock damage (Aires-Barros and Mouraz-Miranda 1989), splitting (Goudie et al 1992) and rockburst phenomena (Ollier 1963).

Topography-related stress: The weight of the overlying rock mass (including pore water and surcharge from mature trees) induces topographic, or gravitational stresses, which increase with depth. The horizontal and vertical components of this stress are related, since vertical stresses will tend to produce horizontal expansion (Selby 1993).

Residual stress: Residual stresses can be a significant contributor to near surface stresses (Nichols 1980) and are defined by Holzhausen (1989, p269) as: ".....those stresses, excluding body forces, that would remain in a regional crustal element if all thermal stresses and stresses applied to its boundaries could be removed". Residual stresses are an internally balanced system of stored inter- and intragranular strains derived from pre-existing gravitational, tectonic or thermal loads (Nichols 1980), or from original crystallisation (Selby 1993). Past stresses become 'locked' into a rock mass at all scales by flow, volumetric change, cementation and chemical alteration (Friedman 1972). They may later become mobilised (Kieslinger 1960; Varnes and Lee 1972) by unloading or the removal of confining pressures; the creation of new rock surfaces; the addition of fluids; and thermal cycling. These mechanisms are brought about by natural processes such as weathering, uplift, denudation, water penetration and diurnal and seasonal temperature fluctuations, or by Mans activities such as rock mass excavation.

6.3.2.2 The nature of rebound

Rebound is defined by Nichols (1980, p133) as: "....the expansive recovery of surficial crustal material, either instantaneously, time dependently or both, initiated by the removal or relaxation of superincumbent loads". Rebound is attributed to unloading, of the type which might occur on a local scale by excavation, for instance, but the processes leading to associated fracture of the rock mass are poorly understood. Rebound fractures have been described by Price (1995) as the first weathering process to act on a newly exposed rock mass, representing the first stage in its progress towards decay. Some rebound fractures may occur at the time of excavation but further fracturing may occur on a time dependent basis. A variety of features can be induced by rebound, and these should not be confused with rock shattering and fragmentation induced directly by the excavation process itself (section 6.3.4.2). The following section examines different types of rebound features induced by rock mass excavation.

6.3.2.3 Features attributed to rebound

(a) Sheeting joints

In his investigation of the granites of New England, Dale (1923, p26) defined sheeting joints as: "...the division of granite into 'sheets' or 'beds' by joint-like fractures which are variously curved or nearly horizontal, being generally parallel with the granite surface". Numerous studies of sheeting joints have been conducted and their features described. The essential features of sheeting joints are reviewed by Twidale (1973) and are as follows:

Sheets are generally parallel with the ground surface (Dale 1923), though not exclusively so (eg Gilbert 1904; Mathes 1930; Jahns 1943; Twidale 1973; Ollier 1984). Sheet thickness, which varies from 30cm to 18m (Dale 1923), increases with depth (Lin and Hung 1995), and the coarser the grain size, the more rapidly the sheets thicken with depth. Sheets have been identified to depths of 30m (Gilbert 1904), and more than 100m (Dale 1923; Twidale 1973), and the depth at which sheets disappear varies with lithology (Jahns 1943). Sheets are laterally continuous, typically 50-60m, and up to 200m in length (Holzhausen 1989). The surfaces of sheeting joints are generally in contact (Holzhausen 1989), although Dale (1923) and Twidale (1973) report 'arching' between planes of up to several centimetres. Sheets have been described as lenticular, with the thick and thin parts of each sheet alternating with those above and below (Dale 1923). Double sets of sheet joints have also been observed (eg Dale 1923). At a microscale, intensification of cracking adjacent to sheets has been observed by Holzhausen (1989). Sheeting joints usually contain very little, if any, infill material.

Sheeting joints are found in all rock types and there does not appear to be any direct link with lithology (Gilbert 1904; Twidale 1973), although they are usually found in fresh, unweathered massive or poorly jointed rock masses (Gilbert 1904; Jahns 1943). Sheets are generally independent of original rock structures such as bedding, foliation and veins (Holzhausen 1989). Sheeting joints may be found in association with shallow localised faults and shear zones (Nichols 1980). They are found in all climatic regions and it is therefore argued that their origin is independent of climatic influence (Twidale 1973). Most sheeting pre-dates glaciation (Jahns 1943) but new sheets are continuing to form in some rock masses and quarry floor spalling is regarded by some (Holzhausen 1989) as being a form of sheeting. New sheeting has also been observed in deep quarries excavated in massive rock (Nichols 1986; Nichols and Collins 1991).

(b) Thin, near-surface laminations

In addition to sheeting joints, much finer 'laminations' (Twidale 1973), or the 'micro-sheeting joints' of Folk and Patton (1982), have been observed to occur parallel to the ground surface (Gerber and Scheidegger 1969), restricted to the extreme upper part of a rock mass which is likely to be susceptible to weathering agents. Such features have variously been described as lift seams, laminations, spalling, rebound joints, flaking, pseudobedding and exfoliation (reported in Twidale 1973). It is clear that weathering agents may be responsible for the continued development of some of these features, but the role of rebound in their formation is unclear. Dale (1923) and Twidale (1973) have suggested that insolation may have a significant role in the formation of thin, near-surface sheets or laminations. These features are more limited in extent, uniformity and depth of penetration than sheeting joints. It is thought that they occur in a wide range of lithology and that they reflect a more complete relaxation of a rock mass than do sheeting joints.

(c) Exfoliation

Ollier (1984) describes various forms of exfoliation manifest as flaking and spheroidal weathering. These are referred to here simply because they have in the past been interpreted as rebound features, and because misunderstandings have arisen concerning the use of the word 'exfoliation'. Ollier (1984, p229) defines the term as: "...a general-purpose word used to describe several processes and several landforms having in common the possession of more or less concentric shells of rock over an inner core". Since on a large scale, such features might be interpreted as rebound fractures, they are mentioned here to underline the distinction.

(d) Spontaneous stress release phenomena

Many occurrences of spontaneous stress release phenomena are a response to artificial excavation of a rock mass. This leads to re-distribution of the pre-existing natural stress field (Gerber and Scheidegger 1969; Lee et al 1979), with concentrations forming at voids, corners and edges (Yatsu 1988). Local stress concentrations in excess of rock strength can lead to spontaneous failure. Numerous examples of spontaneous stress release phenomena have been recorded, including rockbursts (Carlsson and Olsson 1982); spontaneous expansion of quarried blocks (Holzhausen 1989; Nichols and Collins 1991); buckling of quarry floor slabs (Nichols 1980); pop-ups and 'A' tents (Ollier 1984) and a variety of other lateral and vertical rock wall movements (eg Dale 1923; Feld 1966; Gerber and Scheidegger 1969).

Like sheeting joints, rockbursts affect fresh, unweathered, massive rock, and display surface markings comparable to those found on sheeting joints (Holzhausen 1989). Some surface markings resemble those produced in rock following blasting and are evidence of rapid propagation (Holzhausen 1989). Spontaneous stress release phenomena have been attributed to high near-surface compressive stress fields but are more likely to reflect local concentrations of stress due to changes in geometry. The occurrence of spontaneous stress release phenomena in regions where it would be unreasonable to expect the effective transmission of compressive stresses on topographic or geologic grounds, is the basis for the argument by many (eg Kieslinger 1960; Nichols and Abel 1975) that the release of stored strain energy is, in fact, the mechanism responsible.

6.3.2.4 Mechanisms responsible for rebound

Several theories have been advanced for the mechanisms of rebound, particularly those which produce sheeting joints and these are reviewed by Twidale (1973). Some of these theories have largely been disregarded or rejected, including insolation; chemical weathering; plutonic injection; metasomatic expansion; and vertical uplift and will not be discussed further here. In 1904, Gilbert hypothesised on the formation of sheeting joints in granite. He noted that when the granitic magma cooled at depth it was confined and therefore subject to considerable compressive stress, unable to expand. Gilbert envisaged that this compressive stress was balanced by an opposing "expansive stress", which would be sufficient to cause actual expansion of the rock mass if external pressure was removed by erosion of the superincumbent load. This became known as the 'pressure release' hypothesis and was widely adopted (eg Matthes 1930; Jahns 1943). Despite its wide acceptance the mechanisms accounting for this 'expansive stress' were poorly understood. There seems no reason why fracturing should result if rebound is simply the elastic recovery of a rock mass after removal of overburden (Nichols 1980). Twidale (1973) cites a number of further reasons why the concept of 'expansive stress' is inadequate, including the lack of support from experimental work and the contention that, logically, existing planes of weakness ought to take up the relaxation. Several other mechanisms for rebound fracturing are thought possible and these are described below:

(a) Residual stress release

Nichols (1980) argues that release of stored strain energy (ie residual stress) is the mechanism by which rebound occurs and can lead to fracturing. Two stages in the process are envisaged. Initially, an instantaneous, recoverable elastic deformation occurs (Engelder 1984) due to the release of stored strain energy. Subsequently, time dependent relaxation occurs which is inelastic and destructive. This allows penetration of external agents which promote weathering, microcracking and further relaxation. Nichols and Collins (1991) propose that natural excavation of a rock mass by erosion or de-glaciation, for instance, is likely to lead to a large amount of both instantaneous and time dependent rebound, and that because the latter is slow, it will be more complete. For artificial excavation, however, where removal of overburden may be almost instantaneous, the relaxation will be primarily elastic. Some non-recoverable relaxation may also occur in the adjacent non-excavated rock mass (Nichols and Collins 1991). A number of factors influence the residual stress field of a rock mass, including rock strength and stiffness, lithology, chemical composition and hydrology. For instance, rocks which are weaker tend to be less highly stressed. They also weather more quickly and so exhibit inelastic strains more rapidly.

(b) Gravitational stresses

In contrast, Yatsu (1988) favours local stress concentrations to explain rockburst phenomena, and the general distribution of principal stresses in a rock mass in relation to topography to explain sheeting. Yatsu (1988) argues that the axis of minimum stress lies normal to the topographic surface and acts as a tensile stress. This, he proposes, is responsible for topography-parallel splitting. Nichols (1980) argues that the stress ratios involved would be insufficient to cause extensional failure, and that in any case, horizontal or topography-parallel tensile stresses have rarely been recorded.

(c) Compressive stresses

A further mechanism for rebound fracturing involves a purely compressive stress environment. Twidale (1973) suggests three reasons why lateral compressive stress is the cause of sheeting: (i) Sheeting occurs in association with faulting which is often a manifestation of compressive stress; (ii) High compressive stresses have been measured in areas of sheeted rock masses Dale 1923), and evidence of high compressive stresses has been obtained from arched beds (Adams 1982), 'A' tents (Folk and Patton 1982), double sheet joints (Dale 1923) and inselbergs (Twidale 1973); (iii) Experimental work (eg Gramberg 1965; Durelli et al 1968) supports the contention that fractures can develop parallel to a major load axis. In fact, Gramberg (1965) regards axial cleavage fracture as the primary mode of failure, preceding other failure modes in every case of compressive loading of a brittle material.

The various surface markings observed on sheeting planes, including hackle marks, ridge and furrows and slickensides would lend support to the concept of rapid propagation via both axial cleavage fracture and subsequent shear. Lateral compressive stresses sufficient to generate rebound fractures could be derived from tectonic loading or high near-surface differential stresses. If sheeting joints formed only from tectonic compressive stresses, then they should be absent from areas of crustal extension, which they are not (Nichols 1980). However, this may simply reflect a change in direction of palaeo tectonic stresses (Twidale 1973; Holzhausen 1989). Conversely, contemporary tectonic loading may explain the presence of sheeting joints in areas which have never been deeply buried in the past. In a laterally confined rock mass, lateral strain suppression can create a high differential stress field near the surface, even in the absence of any tectonic or other stresses (Herget 1973). Effectively, therefore, a uniaxial stress is present in the horizontal plane, near-surface (Carlsson and Olsson 1982), and values of the same order of magnitude as the uniaxial compressive strength of granites and sometimes greater, have been measured (Holzhausen 1989). The potential to cause fracture is therefore apparent (Holzhausen 1977). If unloading by excavation occurs in a rock mass which is laterally confined or buttressed (Folk and Patton 1982), then relaxation can only occur in the direction perpendicular to the direction of confinement. This mechanism differs from tectonic loading since the lateral stress applied is passive.

Sheeting joints have been observed in areas surrounded by deeply weathered materials which are incapable of transmitting high near surface stresses (Nichols and Abel 1975). It is argued (Nichols 1980) that in areas such as these at least part of the stress field must be created internally due to residual stress. It seems likely that rebound involves a combination of residual stress release, mobilising and concentrating stresses at a local scale *and* compressive stresses operating at a regional scale. These mechanisms might operate independently at some locations, while operating in unison at others. It is equally likely that some of the complexities of interpretation are introduced because of the unknown effects of past stress conditions which are no longer active.

6.3.3 Dynamic stress conditions

Shocks or vibrations either from seismic or artificial activity (eg machinery vibration, blasting operations) constitute dynamic loading and are recognised as potential trigger factors for slope failure (Terzaghi 1950; Brunsden 1979b; Walton 1988; Giani 1992), though their effects on rockslope deterioration and instability are poorly understood.

6.3.3.1 Seismic disturbance

Seismic shock waves resulting from earthquake activity are widely known to trigger rockfalls as well as large deep-seated landslides (eg Keefer 1984; Bull et al 1994; Del Gaudio and Wasowski 2000). Selby (1993) argues that on a global scale, rock avalanches are probably triggered more often by earthquakes than by any other factor. Rockfalls are particularly sensitive to strong ground motion (Bull et al 1994) and Rapp (1960) acknowledges that earthquakes can "prepare and release rockfalls", not only in seismically unstable regions but also in those regions considered stable. He describes the example of Scandinavia, considered a relatively stable sub-continent, where seismic activity can, nevertheless, contribute to rockfall activity. While unconsolidated or weakly cemented deposits and clays are probably most vulnerable to seismic disturbance, steeply jointed rock faces are also significantly at risk (Brunsden 1979b). Rockfall hazard due to seismic activity, however, is extremely difficult to predict.

The dynamic loading which results from seismic activity increases shear stresses in a rock mass and has a tendency to reduce the volume of voids. This causes an increase in pore water pressure, thereby reducing rock strength and resistance to failure. Earthquakes may also result directly in rock fracture. If a rockslope is predisposed to failure, earthquake activity may provide the trigger necessary (Waltham 1994). The effects tend to be amplified with increasing moisture content and at edges and concavities. The distance over which earthquakes can influence slope failure depends on a variety of factors including the magnitude of the earthquake and its focal depth, attenuation properties of the intervening rock mass, and the distribution of shock waves within it (Selby 1993). Earthquakes with a magnitude of greater than 7 have been known to trigger rockfalls 200 to 400km from the epicentre (Keefer 1984). Additional factors which determine the response of a rockslope to seismic waves include the elastic properties of the rock and the geometry of the slope (Blythe and de Freitas 1984).

Research indicates that major rockfalls and landslides occur in response to earthquake magnitudes greater than 4 (Del Gaudio and Wasowski 2000; Nicoletti et al 2000), though failures in unconsolidated deposits have been recorded at magnitudes of 2.9 (Selby 1993). Earthquake magnitudes in the UK rarely reach the levels necessary to trigger rockslope failure. A review of landslides in Great Britain by Lee et al (2000) does not mention the possibility of earthquake induced activity. Nevertheless, low magnitude seismic events occur frequently in Britain and may have a cumulative effect (Selby 1993). It may also be the case that minor failures could be triggered in rock masses with a very low factor of safety by seismic magnitudes less than those indicated.

6.3.3.2 Other sources of dynamic stress

In the context of excavated rockslopes in a UK environment, the most likely other sources of dynamic stress are rock excavation using high explosives and vibration from heavy traffic or railway transport.

The use of high explosives in quarry operations may generate dynamic stresses large enough to affect other slopes in the quarry, including those which have long since been abandoned. Quarry blasting in close proximity to road cuts could also have adverse effects. However, blasting operations of this kind are predictable and designed. This means that the timing of blasts, their frequency and charge strength are known factors. Excavated rockslopes situated in close proximity to blasting operations can therefore be monitored, although the likely effects of dynamic loading of this sort on proposed slopes would be difficult to predict.

It is possible that ground vibrations due to traffic flow or railways may be sufficient to trigger movement on a rockslope, although there do not appear to be any substantive data or investigations which confirm this assertion. However, general principles can be drawn from investigations of the role of traffic vibrations in inducing damage to buildings conducted by the Transport and Road Research Laboratory (Nelson and Watts 1988; Watts 1990). Nelson and Watts (1988) found that ground vibrations from traffic flow were generally not perceptible unless road conditions were poor, containing irregularities from potholes and filled trenches, for example. Measurements of peak particle velocity arising from traffic flow showed an increase with increasing vehicle weight, vehicle speed and road roughness. However, maximum values of 3.5mm/s PPV were recorded at the foundations of buildings situated several metres from heavily trafficked roads (Watts 1990). This compares with a PPV of 10mm/s considered necessary to cause damage to buildings (Figure 6.6). Watts (1990) proposed that ground vibration may nevertheless cause structural damage by (i) providing a 'trigger' in building elements otherwise weakened, or (ii) by fatigue due to prolonged exposure to vibration.

On this basis, it is reasonable to expect that some damage may be induced in rockslopes which are in very close proximity (eg a few metres) to roads on which (i) vehicles move at high speed, (ii) total traffic flow includes a high proportion of HGV's, and (iii) surface roughness is increased by generally poor surface condition. This means that rockslopes adjacent to high speed single carriageway roads, dual carriageways and motorways, particularly where there is little or no verge, may be particularly prone. Quarry slopes adjacent to haul roads may also be vulnerable despite the low speed of traffic flow because of the much greater vehicle weights involved (eg dump trucks) and the relatively poor condition of haul road surfaces.

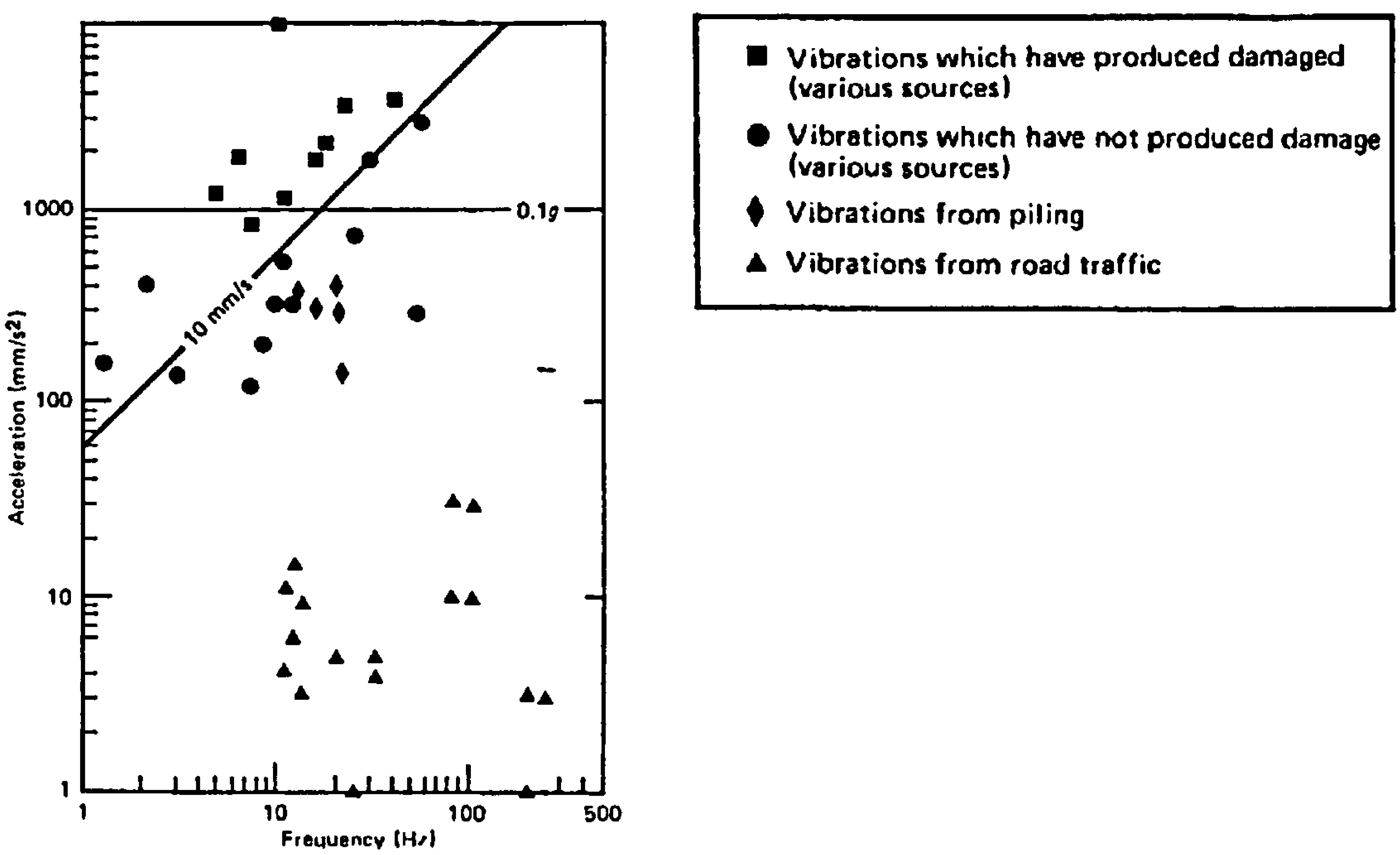


Figure 6.6 Building damage due to vibrations (*after Watts 1988*)

6.3.4 Engineering factors

6.3.4.1 Method of excavation

Natural rockslopes are formed by denudation at relatively low rates over extremely long periods of time (>10^3 years). Agents of erosion include basal undercutting by fluvial and marine action, periglacial processes, landsliding and glaciation. In contrast, the mechanisms of artificial excavation involve the use of tools, machinery and explosives. The rates of excavation in these cases, therefore, varies from months and years to just milliseconds. The selection of excavation method is largely based on rock mass and material properties, though other factors such as relative cost, productivity, planning and environmental impact are also important. Clearly, there is also the fact that with time, new methods of extraction have become available which were not available in the past. In this section, the more common methods of rockslope excavation are described and their potential effects on the rock mass discussed.

(a) Hand and mechanical excavation

Traditionally, highway and railway cuts were excavated by hand, using a combination of wedging and manual extraction of loosened blocks. Occasionally, low explosive (blackpowder) blasting was employed to excavate difficult areas with minimum charges being placed in hand drilled holes. Reviews of the historical development of rock excavation techniques are given by Matheson (1992, 1995). The prime objective for hand excavation methods generally, was to create a rock face which was as steep as could be attained with maximum stability. The survival of most of these extremely steep cuttings to the present day with minimum deterioration testifies to the success of the excavation methods used. In the quarrying of dimension stone, many hand excavation techniques are still in use. The primary objective now is to extract blocks without damaging them in the excavation process. It is usually more cost efficient to extract the largest blocks which can possibly be handled and then to cut them down into blocks of the desired specification size by hand or power sawing. Blocks are often defined by the intersection of joints sets and bedding planes, but where they are not, slots are cut into the rock to define blocks for extraction.

A variety of methods have been used in the past to extract blocks, including wedging, plug and feathers, channelling (drill and broach), sawing and flame jet cutting. Many of these techniques were originally based on simple tools such as hammer and chisel, but modern hydraulic and pneumatic versions have replaced them in many cases (eg hydraulic wedges and splitters, pneumatic rock drills and hydraulic hammers). A variety of powered tools and machinery are also in use, including circular diamond saws, wire saws, chain saws, track mounted cutters and even wheeled loaders with prongs or forks attached for prising out loose blocks. These methods are described and explained in more detail by Smith (1999).

(b) Ripping

In bulk quarrying and in the excavation of cuttings in weak rock, rippers may be used. This involves the use of a crawler tractor with steel tines attached which 'rip' through the rock surface. The material is then scraped or dug up, loaded, then transported away. Ripping is generally carried out in highly fractured, thinly bedded or weak rock. The rippability of a rock mass depends upon the strength of the intact rock and the spacing of fractures, as well as machine specification and performance. For appropriate rock masses, excavation can also be achieved using scrapers, or the buckets of hydraulic excavators.

Several classifications have been developed to assess 'rippability'. Some have been based on fracture index (I_f) and point load strength (IS_{50}) (eg Franklin et al 1971; Pettifer and Fookes 1994), while others have used a wider range of parameters including machine and operator performance to incorporate a prediction of productivity (MacGregor et al 1994). Abdullatif and

Cruden (1983) found that for sites in the south west of England, there was a strong correlation between rock mass quality as determined from the Rock Mass Rating system (Bieniawski 1979) and ease of excavation, and this correlation was better than if only I_f and IS_{50} were used. Seismic velocity has also been used a sole indicator of rippability (Caterpillar 1990).

(c) High explosives blasting

As high explosives became commercially available between 1910 and 1920 (Matheson 1992), blasting became the norm in rock too hard to be excavated using mechanical techniques. Compared to low explosives blasting, larger diameter drillholes are used with a greater charge weight. For non-rippable materials, rock breakage using high explosives is the now the most widely utilised technique for bulk aggregate production and civil engineering projects. Blasting is rarely employed in dimension stone quarrying because of the risk of introducing fractures into intact blocks (see section 6.3.4.2). Blasting involves the placing of an explosive charge, a detonation device and appropriate stemming into a pre-drilled hole, which is then fired. The design of the blast and its subsequent effect on the rock mass involves careful specification of a number of variables, including the number, spacing, burden, angle and depth of drillholes, the amount of charge, the detonation sequence and any delay mechanisms, the number of rows and the type of explosive and stemming used. When the charge is fired, a compressional shock wave travels through the rock and is reflected at boundaries to take up the form of a shear or tensile wave. New fractures may form where the tensile strength of the rock is exceeded. The expansion of gases from the blast causes dilation of fractures, which may partially close again when the gas subsides (Matheson 1985). The rock mass then expands in the direction of the exposed face. The precise role of shock and gas expansion in rock fragmentation is debated (Cunningham 1982). Several types of blast design are used:

Bulk or fragmentation blasting: The primary aim of bulk blasting is to fragment rock to a size which can be handled by plant, and removed to be processed. Little attention is given to the potential for damage to the rock mass by the blasting process.

Pre-split blasting: Pre-split blasting (Figure 6.7) is a form of controlled blasting (Matheson 1995) used primarily for excavation of highway cuts. The primary aim is to minimise disturbance to the excavated slope, thereby minimising maintenance costs and safety hazards. The method enables steeper slopes to be created and therefore is technically and financially attractive (Forster and McGoff 1992).

Contrary to the views of the latter authors, however, pre-splitting creates slopes which are usually unattractive in the aesthetic sense compared with slopes which have been bulk blasted. Pre-splitting involves the drilling of closely spaced holes along the line of the design slope, and these are blasted simultaneously with a light charge. The objective is to create a discontinuity along the design slope. After this, a conventional bulk blast design is applied to the rock mass in front of the design slope. This simply loosens and fragments the rock mass and any excess energy is dissipated into the discontinuity.

The success of pre-split blasting is obviously influenced by the blast design and operation and deviation of drillholes is accentuated because of the close spacing involved. The nature of the rock mass and material properties is also of critical importance. For example, in a very weathered material, drillhole spacings have to be even tighter, using a lighter charge weight (Hemphill 1981). Pre-existing discontinuities and their orientation relative to the pre-split plane also influence the final result. Gas can expand along these fractures, for instance, allowing fracture propagation to occur in a direction which is not consistent with the design slope plane. This can produce an irregular face with areas of underbreak and overbreak (Matheson 1983b).

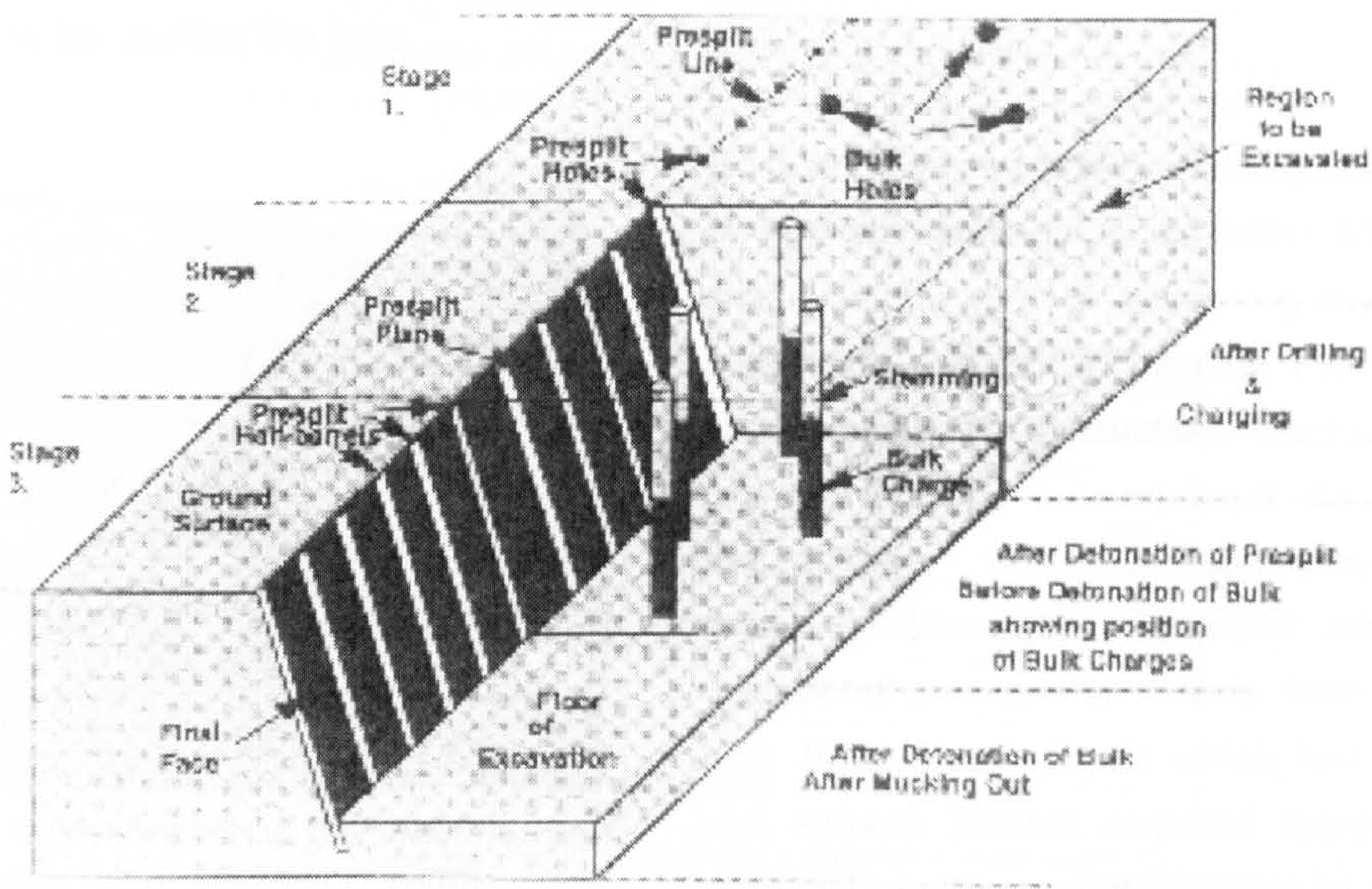


Figure 6.7 A three stage representation of pre-split blasting (*after Matheson 1995*).

Smooth blasting: Smooth blasting is also a type of controlled blasting, but differs from pre-splitting in that the contour charges are fired *after* the bulk blast. The spacing of holes is typically 600-1000mm compared with 500-800mm for pre-splitting (Romana 1993) and is therefore less expensive. Successful smooth and pre-split blasted faces have been developed in a range of lithologies, reaching face heights of up to 62m (Matheson 1995).

Block production: Blasting may also be undertaken with the aim of obtaining a particular block size specification, for armourstone for instance. In these cases, a small charge of low explosive (blackpowder) may be used, and widely spaced drillholes located along existing discontinuities (Smith 1999). The desired production block size can be obtained from careful blast design if the characteristics and Natural Block Size Distribution (NBSD) of the rock mass is known (Langefors and Kihlström 1978; Wang et al 1992).

6.3.4.2 Blast damage

Blasting involves instantaneous application of high stresses to a rock mass. In addition to causing the designed fragmentation of rock, this can also damage the retained rockslope. Matheson (1985) and Swindells (1985) have investigated the damage caused to a rockslope by different types of excavation and were able to assess the likely width of damage zone in each case. Matheson (1985), using pre and post-blast scanline surveys of the slope, down-hole CCTV and surface refraction surveys, determined that for all types of excavation (natural, hand excavated, pre-split, smooth and bulk blasted) there was a zone up to 1m width near the surface in which there was a small increase in *fracture intensity*. In fact, the biggest effect of excavation was the *dilation of existing fractures* (Figure 6.8). Matheson found that natural slopes, and those excavated by hand or by pre-split blasting, displayed no zone in which fracture dilation was evident, smooth blasted slopes had a zone 4m in width, and for bulk blasted slopes the damage zone was 2 to 8m. These values are comparable to those of Swindells (1985). In all cases, from visual observation, slopes with the widest damage zone were also the least stable. It is interesting that the zone of greatest fracture dilation in the bulk and smooth blasted slopes was situated 1 to 2m *behind* the face.

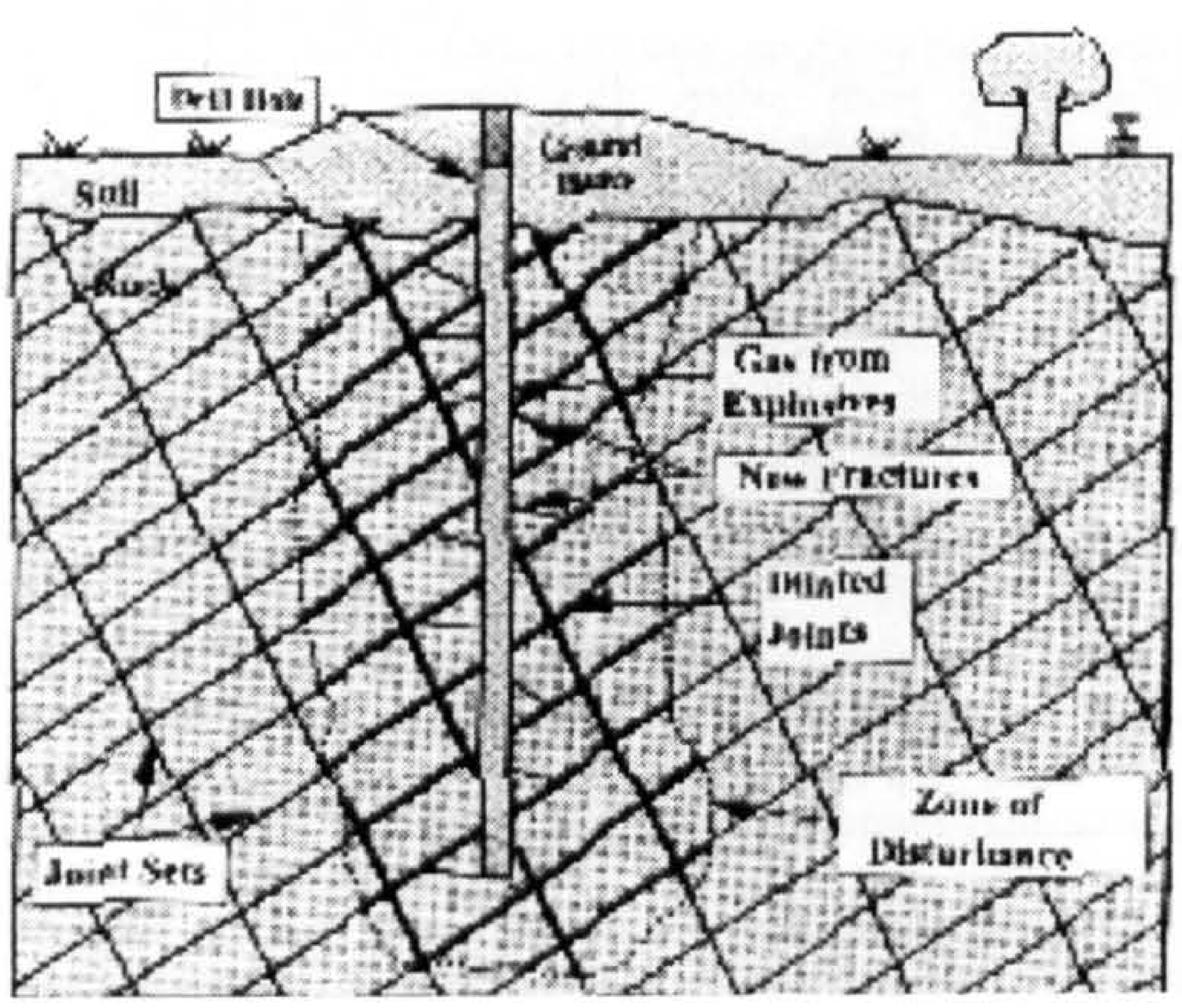


Figure 6.8 Idealisation of the blast damage zone (*after Matheson 1995*)

Ross and Reeves (1995) investigated a number of highway cuttings in andesite and dolerite, also with the aim of assessing the effect on stability of different excavation methods. They found that hand excavation, blackpowder and pre-split blasting caused least disturbance to the slopes, which subsequently required minimal maintenance, localised dentition and occasional removal of debris from the slope foot. In contrast, slopes which had been bulk and smooth blasted required more extensive remedial measures, containment by netting and barriers, and extensive scaling.

It is evident that the development of new, open fractures and the loosening effect of the rock mass due to dilation of existing fractures increase the potential for deterioration and instability. To reflect this, Romana (1988, 1993) incorporated a factor in his Slope Mass Rating (SMR) system to reflect the method of excavation used. He distinguishes six categories of excavation: natural (+15), pre-split (+10), smooth blasted (+8), bulk blasted (0), mechanically excavated (0) and deficiently blasted (-8). The values in parenthesis are the adjustment factors applied in each case, where a rating of 20 constitutes a whole class difference. The justification given for these rating adjustments is that pre-split and smooth blasting *improve* slope stability, bulk blasting and mechanical excavation do not alter slope stability, and deficient blasting damages the rock mass and therefore reduces stability. Natural slopes are regarded by Romana as being inherently stable due to their slow rate of excavation and the protective effect of surface cover such as vegetation and crust. The ratings are based on depths of disturbance zone reported by Swindells (1985).

6.3.4.3 Engineering design factors

Engineering design factors which can influence rockslope deterioration include slope geometry, stabilisation measures and ongoing maintenance works. These are considered further below.

(a) Geometry of the slope

Slope geometry incorporates a number of elements, each of which may have an influence on deterioration. The most important elements are summarised in Table 6.1.

Orientation
• Orientation determines slope aspect and therefore strongly influences climatic input to the slope; • Orientation controls the intersection of discontinuities with the slope face and therefore has a major influence on stability.
Slope gradient
• Runoff velocity is increased on steeper slopes; • The relationship between slope gradient, dip of major discontinuities and friction angle is critical to slope stability; • In relation to aspect, gradient determines the angle of incidence of solar radiation; • Large plants are less able to establish on steeper slopes.
Slope height
• Greater slope height increases the risk of deterioration due to direct impact from falling blocks; • Greater slope height (perhaps due to surcharge) reduces stability due to extra overburden load; • Greater height may also increase the hazard resulting from the fall of material because of the greater likelihood of bouncing, rolling and throw.
Slope length
• Surface runoff volume and velocity is increased on longer slopes thereby increasing erosion potential.
Slope morphology
• Internal stress may be concentrated at sharp changes of slope angle, boundaries and other irregularities; • Micromorphology affects microclimate and local slope gradient, and can, therefore, influence local vegetation establishment (eg in sheltered hollows), soil accumulation and water retention; • Undermining of competent strata by weathering and erosion of weaker materials, or basal undercutting in homogeneous materials, can produce overhangs which are vulnerable to collapse.

Table 6.1 Elements of slope geometry and their influence on deterioration

(b) Slope stabilisation and protective measures

A variety of rockslope stabilisation measures are discussed in Chapter Eight. As a bi-product, however, some remedial measures have the potential to act adversely on the rock mass and some examples are considered here. (i) Rockslopes may be selectively soiled and vegetated both for aesthetic reasons and to provide a surface protective cover. However, a moist soil medium may enhance chemical weathering and plant roots may exploit rock fractures causing physical breakdown. (ii) The heads of rockbolts may provide a micro-catchment for surface water collection which can lead to granular disintegration. In time, this may create a weathered channel behind the bolthead, reducing its effectiveness. (iii) If a drainage pipe is covered with a masonry structure for aesthetic reasons, any subsequent failure of the pipe can lead to internal and hidden build-up of water pressure. If unnoticed, this may eventually develop into a seepage from the face, at the site of which, erosion or enhanced weathering may occur. (iv) Scaling to remove loose blocks may have the effect of exposing further loose blocks deeper in the rock mass. This can be particularly problematic if the scaled area leaves an overhang. (v) Materials used in stabilisation measures such as masonry, cobbles, concrete and shotcrete may themselves be susceptible to deterioration.

(c) Ongoing maintenance

The very action of maintaining a slope or installing new remedial works may disturb the slope and re-activate deterioration. Furthermore, the removal of weathered and loose material exposes fresh, stable rock to the weathering environment and may render it vulnerable to attack.

6.3.5 The influence of exposure time

Rock weathering and erosion rates are a function of many factors including rock mass and material properties, climatic, geomorphic and stress conditions. Deterioration rates, therefore, vary considerably from one location to another even where one or more of these variables is identical. It is often difficult, therefore, to determine the exact explanation for variations encountered. Natural erosion rates have been calculated from the accumulation of soil, from studies of karst water chemistry (Groom and Williams 1965), cliff recession rates (Ollier 1984) and isotope analysis. For denudation of natural landforms, weathering rates in the order of millimetres or centimetres per thousand years are usually measured. Investigation of dated historic stone buildings and monuments (eg Emerick 1995; Robinson and Williams 1996) also reveals significant, measurable weathering over a timespan of up to a few hundred years, while exposure trials indicate significant rock weathering occurs over decades (Hilger 1897, reported in Ollier 1984) or even months (Carter and Viles 2000).

The premise of this thesis is that deterioration of rockslopes is *accelerated* due to excavation and that very substantial deterioration can occur in *engineering time*, say over 50 to 100 years. Investigations of deterioration rates for man-made landforms are rare. Gagen (1988) showed that in disused limestone quarries, substantial karst-like sinkhole features had developed since quarrying began 100 to 150 years ago, and that these were the primary agent of face recession by frequent rockfalls. He determined that for some limestone quarries, recession occurred in successive stages beginning with initial rockfalls from blast fracture cones almost immediately after excavation. Significant slope collapses occurred between 2 and 10 years after excavation, eventually leading to the development of buttresses and headwalls which then coalesced after 25 years. Gagen's work (1988) showed that increasing time since excavation in a single quarry correlated with higher magnitude events, with more frequent and smaller magnitude rockfalls occurring on younger faces. Several workers including Douglas (1980) and McDonnell (2000) have established an inverse relationship between magnitude and frequency of mass movements from natural rockslopes (Figure 6.9). However, attempts by Gagen (1988) to establish the same relationship across a range of different

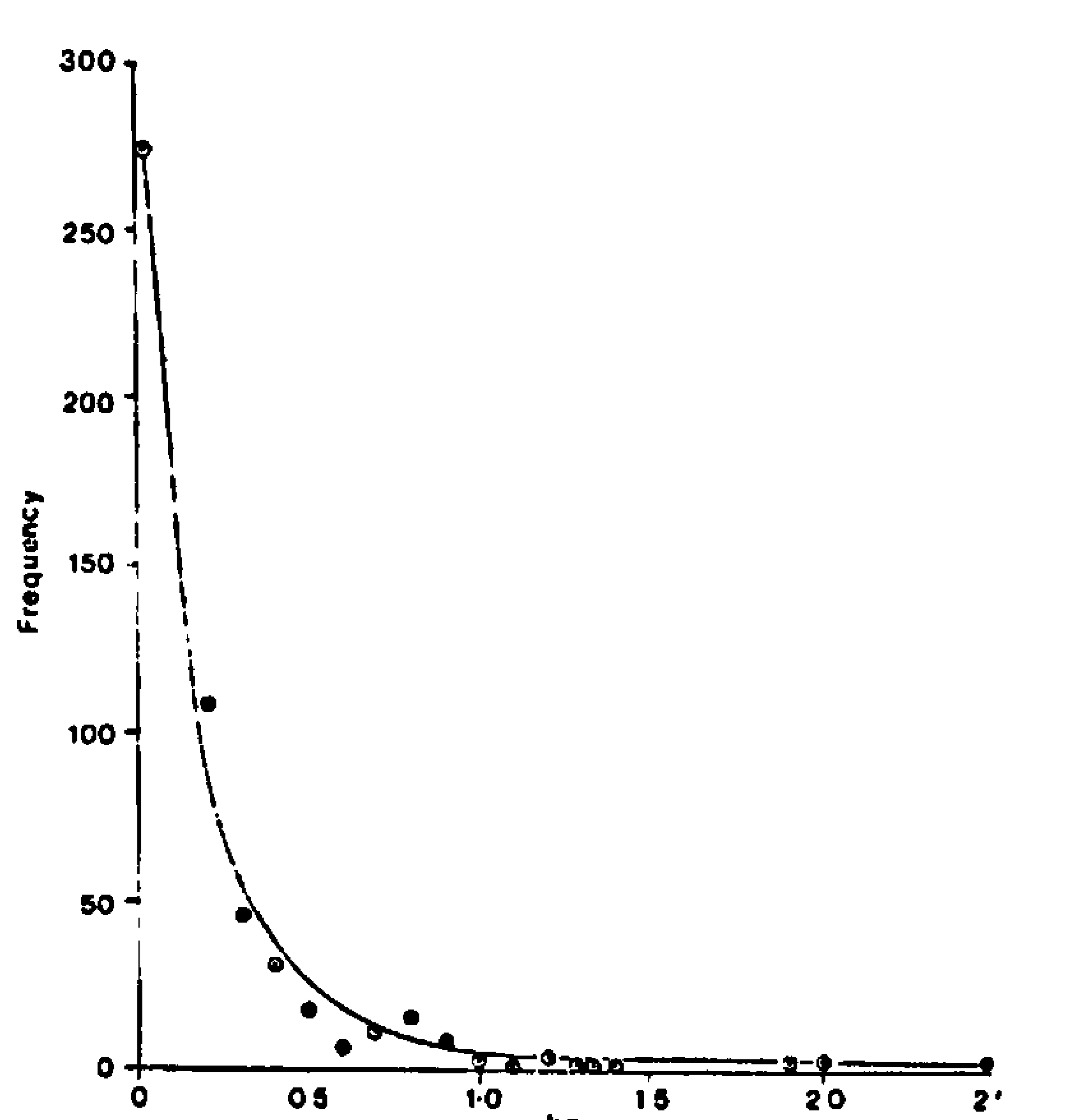


Figure 6.9 Magnitude frequency relationships established from rockfall activity in County Antrim (*after Douglas 1980*)

quarries was unsuccessful. Field observations reported here also indicate a higher frequency of falls of material on younger rockslopes, but high magnitude events were not noticeably more likely on older slopes. It is not clear to what extent correlation between type, magnitude and frequency of rockfalls is simply a reflection of changes in excavation methods adopted over that time period. Relationships between magnitude and frequency clearly relate to identifiable fall or collapse mechanisms rather than in situ weathering. The latter can be regarded as time dependent, occurring semi-continuously and progressively over time. Clearly disturbance by Man or by unusual trigger factors such as seismic activity, may override the equilibrium status of the rock mass.

6.3.6 Triggers for rockslope deterioration

Deterioration of rockslopes, while influenced and controlled by the range of intrinsic and external factors described above, may also require a trigger. Several authors have adopted the concept of factor of safety in which movement of material on a slope is triggered by either (i) an increase in stress due to external changes (eg changes in geometry, dynamic stress, increased pore water pressure), or (ii) a reduction in rock mass strength due to internal changes (eg weathering and erosion) (Terzaghi 1950; Brunsden 1979a; Hansen 1984). In the context of deterioration this concept is over-simplified since it envisages a moment of 'failure'. This may be absent where time dependent material weakening occurs due to in situ weathering, regarded in the factor of safety approach as a trigger factor, but in the context of deterioration as a failure mechanism in its own right. Nevertheless, the idea that certain factors can trigger the fall of material from a slope is a useful one. It helps to explain the temporal distribution of rockfalls (Figure 6.10), for example, with peak frequencies in spring and autumn thaw periods (Rapp 1960; Douglas 1980).

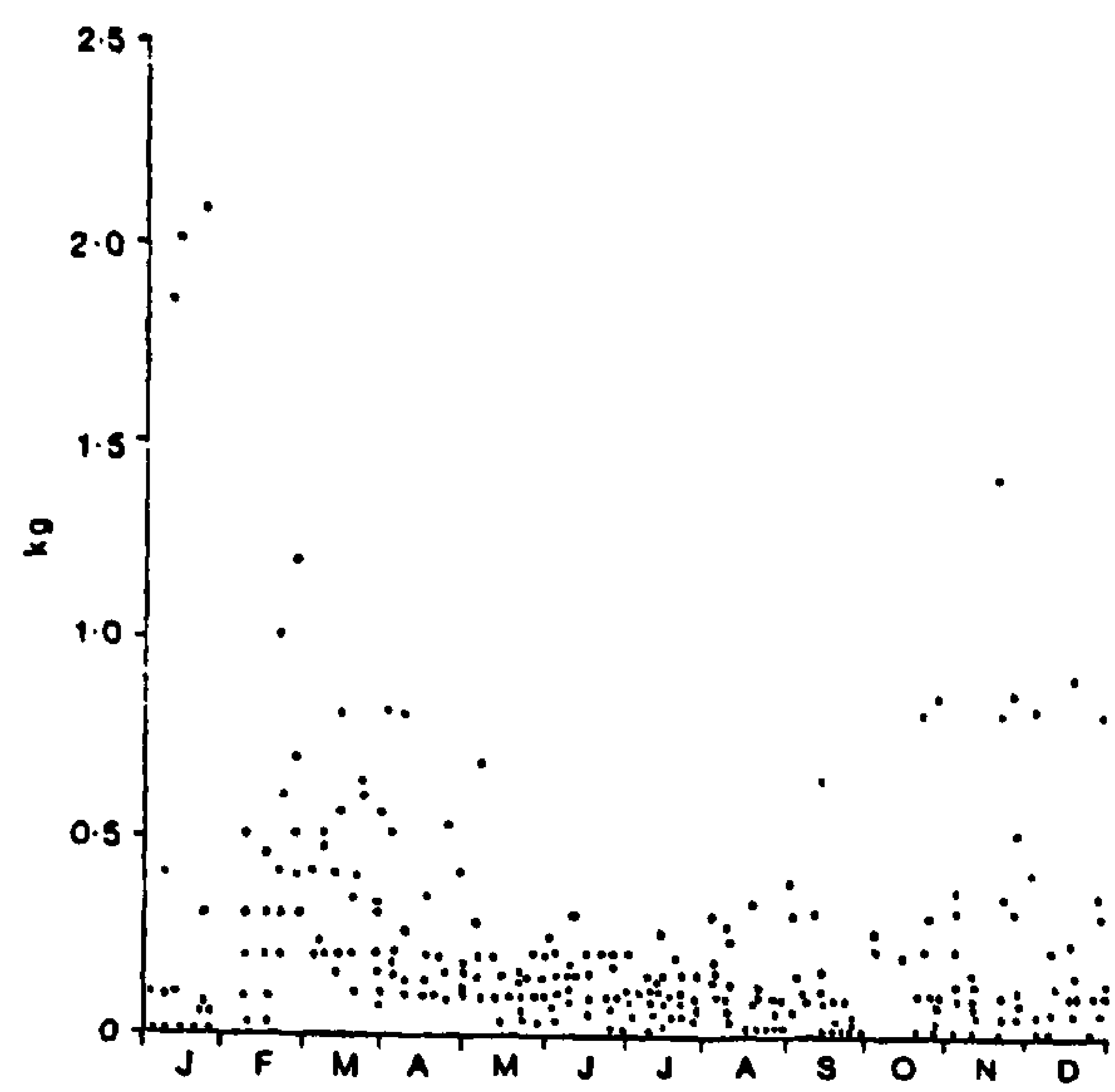


Figure 6.10 Temporal variation in rockfalls from basalt cliffs in County Antrim (*after Douglas 1980*)

CHAPTER SEVEN
RESULTS OF THE FIELD INVESTIGATION OF ROCKSLOPE DETERIORATION

7.1 General Aims of Fieldwork

A key aim of the research was to develop a systematic procedure for the assessment of excavated rockslope deterioration. However, it was first necessary to establish a basis for the description, classification and assessment of deterioration, particularly in view of the general lack of published data pertaining to this subject. The objectives of the field investigation therefore, were (i) to establish the extent of deterioration for excavated rockslopes in the UK; (ii) to characterise the nature of deterioration; and (iii) to determine how it can be recognised on *existing* slopes. Evidence of the consequences of deterioration were also sought, as were influencing and controlling factors. A greater knowledge of the latter might enable prediction of the likely occurrence of deterioration for *proposed* rockslopes.

7.1.1 Data collection

Three main types of data were collected for the field investigation: (i) Factual, published and anecdotal information derived from others (section 7.2.2); (ii) qualitative observational data (section 7.2.3); and (iii) quantitative data (section 7.2.4). In deciding what data to actually collect, reference was made not only to the review of influencing factors given in Chapter Six but also to the practicalities of data collection in the field, the availability of suitable equipment and ease of use. The final selection of data parameters and methods of data collection are considered in section 7.2, before the results of the field investigation are given in section 7.3.

7.2 Field Investigation: Site Selection and Data Collection

7.2.1 Site selection

In the field investigation Man-made rockslopes exceeding 45° in gradient were considered. Soil, soil-like and unlithified materials are excluded except where they form part of a slope substantially cut in rock. A wide range of rock types is represented, though in the UK there was inevitably an emphasis on rocks of sedimentary origin. Attempts were made to include a wide range of metamorphic and igneous rock types but because of their limited geographical distribution some non-sedimentary rocks are absent from the survey. Sedimentary rock types which rarely form cut slopes (eg evaporites and some rocks of organic origin) are also absent. Many of the slopes assessed are located in the north of England, there being a notable emphasis on locations in North and West Yorkshire and Cumbria. Other slopes have been investigated as far afield as central and northern Scotland, Devon, Sussex, North Wales and Northern Ireland. A very small number of non-UK slopes were also investigated primarily because there were features of particular interest to be explored and because opportunities became available. Because of climatic differences though, the applicability to UK conditions of these results is questionable.

SEDIMENTARY ROCKSLOPES	
Total number of slopes	51
Total number of slope units	110
Slope type	Road cutting = 48; disused quarry = 57; semi-active quarry = 8; active quarry = 1; natural slope = 3
Rock type* *The total number of rock types exceeds the total number of slope units since some slopes comprised more than one rock type.	Sandstone = 62 (including 10 gritstone, 4 calcareous sandstone, 1 flag and 1 turbidite). Limestone = 43 (including 9 oolitic limestone; 8 chalk and 1 magnesian limestone). Mudstone = 25 (including 8 siltstone and 8 shale). Breccia = 2.
IGNEOUS ROCKSLOPES	
Total number of slopes	19
Total number of slope units	47
Slope type	Road cutting = 16; disused quarry = 29; semi-active quarry = 0; active quarry = 1; natural slope = 1.
Rock type* *The total number of rock types exceeds the total number of slope units since some slopes comprised more than one rock type.	Granitic = 24 (including 12 microgranite, 8 pegmatite and 4 granite). Basaltic = 17 (including 9 basalt, 5 dolerite and 3 pillow lavas). Pyroclastics = 14 (including 11 tuffs and 3 ignimbrite).
METAMORPHIC ROCKSLOPES	
Total number of slopes	27
Total number of slope units	53
Slope type	Road cutting = 37; disused quarry = 13; semi-active quarry = 0; active quarry = 3; natural slope = 0.
Rock type* *The total number of rock types exceeds the total number of slope units since some slopes comprised more than one rock type.	Metasediments (metamorphosed turbidite sandstones, siltstones and mudstones) = 22. Gneiss = 18. Slate = 8. Schist = 7. Phyllite = 2.

Table 7.1 Summary details for sites investigated

The total number of slope units investigated was 210, based at 97 sites. In this context, slope units are simply limited sections of a slope made distinct from adjoining units by a significant change in rock mass or material properties, or some external influence such as aspect. Many of the slopes investigated consist of a single slope unit, while others consist of up to eight units. Of the slopes investigated, 46% were situated in disused quarries and 46% were road cuttings. The remaining slopes were active quarries (2%), semi-active quarries (4%) and natural rockslopes studied for comparative purposes (2%). More detailed information on sites investigated is given in Table 7.1 above.

7.2.2 Factual, published and anecdotal information derived from others

This included (i) Information from published maps and documents: Examples include site location (grid reference) and name; altitude; geological formation and age where known. (ii) Information from landowners, highway authorities and other authorities (eg quarry operators): Examples include excavation, treatment and maintenance works; hazards or known problems; time since excavation. (iii) Other miscellaneous information sources: example include survey and inspection reports, borehole logs, historical photographs, and on-site discussions with engineers.

7.2.3 Qualitative observational data

A key objective of the field investigation was to characterise deterioration and its potential consequences. Direct and indirect evidence of the following items was recorded:

The general nature and distribution of deterioration: Deterioration modes, magnitude, event frequency and spatial variation within rockslopes were recorded. Since the geographical and lithological distribution of the slopes investigated was also known the spatial distribution of deterioration could also be determined. A note was made of any correlation between spatial variations in deterioration and corresponding rock mass and material properties.

The consequences of deterioration: Evidence of stabilisation measures, maintenance works and failed remedial treatments was recorded, as well as the interaction between these and observed deterioration. The implications of deterioration for safety, maintenance, remedial measures and other issues (eg aesthetic impact, slope recession and geological conservation) were also recorded where known or where obvious.

Deterioration processes: Direct or circumstantial evidence of weathering processes and erosive agents was recorded, such as the presence and distribution of weathered scars, areas of surface dampness or groundwater flow, and the presence and general type of vegetation.

The effects of deterioration on slope morphology: Erosional and depositional landforms and indications of in situ weathering were described, and in some cases, sketched and photographed. Features included slope morphological forms such as overhangs, chutes and debris piles. The general dimensions of such features and their constituent materials were also made.

7.2.4 Quantitative data

A range of measurements and observations were made of quantitative parameters including various rock mass and material properties. Observations made and the methods used are described in this section.

7.2.4.1 Rock material properties

Grain size was measured visually using a x10 hand lens where necessary. The grain size classification recommended in BS 5930:1999 was adopted, where clay < 0.002mm; silt = 0.002 to 0.06mm; fine sand = 0.06 to 0.2mm; medium sand = 0.2 to 0.6mm; coarse sand = 0.6 to 2mm; and gravel > 2mm. ***Texture and fabric*** were assessed visually using standard geological terms such as granular, crystalline, vuggy, thinly laminated and banded as appropriate. ***Colour*** was assessed visually using terms similar to those recommended in BS 5930: 1999. ***Weathering grade*** was assessed visually and with the use of a geological hammer. A slightly modified form of Moye's (1955) granite material weathering classification was used and is reproduced in Chapter Eight. ***Rock strength*** was assessed in two ways. A geological hammer or hand breakage was used to provide a field estimate of rock strength in accordance with the scheme recommended by the Geological Society Engineering Group Working Party (1977). At some localities, an N-type Schmidt hammer rebound value was also obtained using the method recommended by Poole and Farmer (1980). Other properties were recorded as appropriate. These included field estimates of cementation and porosity where possible, and mineral composition. ***Rock type*** was evaluated on the basis of the above observations, and named broadly in accordance with the scheme of Norbury et al (1986).

7.2.4.2 Rock mass properties

Fracture spacing was measured along horizontal and vertical scanlines up to 10m in length held along the slope face. Contrary to the ISRM (1978b) recommendations, *every* fracture (ie mechanical break) and incipient fracture was included in the total. So all fractures induced by blast damage, weathering, stress relief and anthropogenic activity were included since deterioration does not distinguish fracture origin! Mechanically intact discontinuities such as bedding planes and other lithological fabrics were excluded. At many localities, short scanlines were used as an initial guide to fracture spacing and this was subsequently transposed to mean block size by visual judgement. In accordance with the ISRM (1978b), typical minimum, maximum and mean block dimensions were recorded. In most cases it was possible to estimate three-dimensional block size but in others block size relates to two-dimensional traces only. ***Fracture persistence*** was recorded on the basis of visual judgement using a simple, scale of very persistent, persistent, sub-persistent and non-persistent, each category being related to the scale of the rockslope. So for example, a 2m joint on a 20m high slope would be described as sub-persistent, but the same joint on a 5m high slope would be regarded as persistent. ***Fracture origin*** was recorded where discernible. ***Orientation*** was recorded for each distinctive fracture 'set'. For some joints and bedding planes it was often possible to measure a typical dip and dip direction for the set. For other fracture 'sets' such as random blast-induced fractures, general descriptors were use such as randomly distributed, angular, curved, irregular and lens-shaped. ***Fracture aperture*** was measured perpendicular to the fracture trace either with a calliper (measuring to 0.1mm for apertures up to 1.5mm) or standard ruler calibrated in millimetres (for apertures exceeding 1.5mm). ***Infilling*** material was examined visually, sampled for simple hand texture analysis where possible, and described. Key descriptors related to grain size, texture and organic matter content. The possible origin of infilling material was also recorded since this might indicate weathering processes. Vegetative infill was also recorded. No attempt was made to discern the chemical composition of infilling materials though it was recorded where obvious (eg iron oxide deposits).

Rock mass structure was described on the basis of visual judgement and comprised two key elements. The first was a geometric description of the typical block shape and broadly follows the ISRM (1978b) terms (eg massive, blocky, layered, columnar and irregular). Additional terms were introduced as necessary, including fissile, composite, intensely fractured, karstic and rubbly and these are defined in section 7.3.5.2. A visual evaluation of the 'looseness' of the rock mass was also recorded. The second element was a geological description of the rock mass structure. This utilised standard geological terms such as *bedded* and *structureless.* The spatial distribution of features within the rock mass was also noted, including unconformities, buried channel deposits, igneous intrusions, contact zones and cavities.

7.2.4.3 Environmental factors

Exceptional climatic conditions were recorded on the basis of visual judgement, such as location in a frost pocket (eg permanently shaded, exposed areas) or subject to cold air drainage. ***Aspect, altitude*** and ***vegetation cover and type*** were also noted. The degree of ***shelter and exposure*** afforded to slopes was described with reference to surrounding topography, shelterbelts, structures or land. Some hypothetical examples based on real observations are given below.

- A two-lane highway with excavated rockslopes on each side of the road would provide a ***sheltered*** environment, notwithstanding any other extremes of altitude or climatic conditions. Alternately, a continuous, multiple layered belt of evergreen trees opposite the rockslope, reaching to at least the height of the slope and densely underplanted with evergreen shrubs would provide a sheltered environment. The distance between the shelterbelt and the slope would need to be less than about twice the height of the slope. A discontinuous, or single layered, or deciduous belt of trees would provide less shelter. Other sheltered environments would be afforded by a location in a slot-like quarry or by being surrounded on all sides by higher land in close proximity.

- A medium sized quarry enclosed on all sides might typify a ***slightly exposed environment.*** It is difficult to define 'medium sized' since to some extent the level of shelter afforded depends not only on the length and breadth of the quarry, but on the relationship of this dimension to its depth. A dual carriageway with cuttings on either side would be another example.

- A ***moderately exposed*** slope would be a very large quarry where the presence of slopes effectively enclosing the site does not afford any sheltering effect. A motorway crossing open land which passes through cuttings would be another example, as are slopes which face onto open land which rises up around 1km distant.

- Slopes which could be regarded as ***very exposed*** would be those situated on topographic highs at relatively high altitude overlooking land which falls away into the distance, with no shelter provided, and completely exposed to the elements. Individual units of a such slopes which were locally protected by close proximity vegetation, structures or other slopes might be classed only as moderately exposed.

Exposure levels can be raised by increasing altitude or latitude.

Groundwater seepage and surface water runoff observations were broadly based on the scheme recommended by the ISRM (1978b). Several descriptive categories are possible: no seepage (rock dry, no evidence of seepage); very minor seepage (localised slow dripping or surface dampness); minor seepage (steady to fast dripping and larger areas of damp or wet rock surface); moderate seepage (light, continuous flow and large areas of wet rock); major seepage (continuous flow); excessive seepage (continuous, strong flow). Because it was not usually possible to undertake site investigation at times of strongest seepage flow, seepage often had to be inferred from indirect evidence.

7.2.4.4 Static and dynamic stress conditions

Observations made pertaining to static stress conditions related to slope geometry and situation. Slope height was recorded or estimated and the general form of the slope described. A note of any surcharge loading at the crest was made where applicable and the amount of overburden present or removed was made.

The presence of any close proximity blasting or other vibration-inducing machine operations was noted. The nature of traffic flow, in terms of volume, speed and the approximate proportion of heavy vehicles was noted where appropriate. These observations were necessarily estimates, but some idea could be gained from the nature of traffic flow at the time of the visit(s), as well as the nature of the site (eg active quarry, disused quarry, highway), and in the case of roads, the class of road, its location, condition and key destinations.

7.2.4.5 Engineering factors

Descriptions were made of any stabilisation or protective measures present either on the face (eg rockbolts, netting, shotcrete, dentition, drainage), at the slope crest, where accessible (eg crest drainage, cabling), at the slope foot (eg toe drains), or in the immediate vicinity (eg rocktrap ditch and fencing). Special note was made of any evidence of deterioration directly related to, or even caused by such works. Evidence of damage caused to structures by deterioration was also noted. In addition to slope form and height, slope angle and the approximate length of the slope were also recorded. The presence of any undermining or active erosion at the base of the slope were noted.

7.2.4.6 Time since excavation

Where known, the time since excavation was recorded or obtained. Some data in this category are accurate while others are estimates with an error margin as great as ±20 years and even greater in a few cases. Inevitably, less accurate data are available for older slopes, though this is not necessarily always the case.

7.2.5 Data collection proforma

A data proforma (similar to that shown in Figure 8.3) was used for the field investigation.

7.3 Results of Field Investigation

7.3.1 Deterioration: occurrence, consequences and mitigation

From the field investigation conducted it is apparent that deterioration is widespread on excavated rockslopes in the UK. The nature of deterioration in terms of the volume and type of constituent material, the frequency of fall 'events' and the mechanisms involved, varies considerably. As a result, the consequences of deterioration, the approaches adopted for its mitigation and the stabilisation and protective measures used, also vary.

All forms of deterioration were represented, including occasional fall of individual grains resulting from in situ disintegration, semi-continuous ravelling of highly fractured rock masses, and isolated rockfalls involving large volumes of material. Deterioration occurred to some extent on every rockslope investigated, though in some cases was inconsequential. Many of the potential consequences identified in Chapter One were observed, including damage to structures (Plate 7.1), debris on the road pavement (Plate 7.2) and over-topping of catch ditches (Plate 7.3). For many highway slopes the safety hazard and maintenance burden arising from deterioration depended on proximity to the road as much as to the nature of deterioration involved. Extensive deterioration of the M6 Dillicar cutting, for instance, is largely irrelevant because of the extremely wide grass verge between the foot of the slope and the hard shoulder crash barrier. Conversely, deterioration of the A170 at Sutton Bank can be perilous due to a very narrow verge which is as little as 0.5m in places. Here, the maintenance burden is high in the sense that frequent inspections and twice yearly scaling must be undertaken in order to minimise safety risks (Plate 7.4). Maintenance is less of an issue in active quarries except where haul roads pass close to a deteriorating face. Safety is more critical here, where quarry workers might be injured by falling material. For active quarries, the inspection and assessment of slope stability is addressed in the new Quarry Regulations (Health and Safety Commission 1999) and will be undertaken in accordance with this legislation. However, at disused quarries, safety is perhaps more of an issue since in addition to the risk of injury to visitors, damage to structures and property can be caused where quarries are now occupied by industrial developments, caravan parks and educational and recreational facilities.

Plate 7.1 Structural damage on the A6 at Wasdale Beck, Cumbria (*left*). Impact marks on the concrete drainage channel are evidence of the role of falling material in structural damage.

Plate 7.2 Encroachment of debris onto the road pavement at Southerham Industrial Estate, Lewes, Sussex. Widespread ravelling of this chalk slope necessitates frequent debris clearance. Fortunately, the road is a minor access road in an industrial estate and thus is lightly trafficked.

Plate 7.3 Overtopping of a catch ditch on the A58(T) at Godley Cutting, Halifax, West Yorkshire. Semi-continuous flaking of this shale and mudstone slope leads to rapid filling of the rock trap ditch. Frequent clearance is necessary to reduce the risk of material overtopping the wall and presenting a serious hazard to road users.

Plate 7.4 Maintenance works on the A170(T) at Sutton Bank, North Yorkshire. The very narrow verge at the side of the heavily trafficked road means that regular maintenance is required. There is no scope for slope re-design, and treatment measures are restricted because of the location in a high quality, sensitive landscape.

Plates 7.1 to 7.4 Some of the consequences of rockslope deterioration

In section 1.2.3 of Chapter One, reference was made to the possible consequences of deterioration arising from morphological change to the rockslope itself (ie boundary modification, aesthetic impact and conservation issues). Several examples were observed in the field investigation where exposures at Regionally Important Geological and Geomorphological Sites (RIGS) and other localities described in field guides (eg Moseley 1990; Cumberland Geological Society 1992) were partially concealed as a result of deterioration and thus their value was reduced. Other guides (eg Mortimore 1997) warn visitors of the necessity for safety precautions as a result of face deterioration. There was an inverse correlation between deterioration and adverse aesthetic impact. Slopes which had developed a weathering crust, or had become stained by water flow, or had an irregular surface topography, or which did not show evidence of their artificial origin (eg drillholes, stabilisation measures) or which had a good cover of vegetation, were the least visually intrusive landforms. Conversely, freshly excavated slopes in which stabilisation measures had been used extensively or where pre-split blasting drillholes were in evidence, often presented a negative aesthetic impact, though this of course varied with the nature and quality of the surrounding landscape (Nicholson 1995).

In general, however, the author was impressed by the efforts which had been made on highway slopes to minimise visual impact and to blend stabilisation measures in with the rock material. For instance, several examples were seen of shotcrete which had been dyed to the same colour pigment as the rock in order to camouflage its presence. The dry stone retaining wall constructed at the Beamsley cutting on the A59 (Plate 7.5) is also impressive.

From discussions with highway engineers and inferences made on the ground, it is apparent that in most cases, deterioration is dealt with on an ad hoc basis, usually with no systematic inspection or maintenance visits being conducted until specific problems are reported. Once such a problem has been identified a common regime for more severely deteriorating slopes is for site inspections to take place twice annually in spring and autumn. The perception is that greater freeze-thaw activity occurs during these times leading to greater deterioration. Maintenance visits are conducted on an as-needed basis thereafter.

Maintenance might simply involve cleaning out of catch ditches and drains, collecting data from slope instrumentation and repairing stabilisation measures, rather than any active work on the rock face itself. In other cases, maintenance involves scaling the face by hand, using a hydraulic lift to access higher parts of the slope where necessary (Plate 7.4). A moderate number of highway slopes had stabilisation and protective measures installed. Those measures most commonly in evidence were wide verges with or without a protective fence or barrier; wire mesh netting (Plate 7.6); rocktrap ditch and fencing; rockbolts and dowels (Plate 7.7); shotcrete (Plate 7.8); masonry buttressing (Plate 7.9); local dentition; and toe and crest drainage. More rarely, metal retaining strips were used (Plate 7.7), as well as cabling, and concrete buttresses and retaining walls.

Plate 7.5 Innovative slope retention on the A59 at Beamsley, West Yorkshire (*left*). Large blocks obtained from the excavation have been used to form a dry stone retaining wall. This is not only an effective stabilisation measure, but has been constructed in such a way as to blend in with the natural geological character of the in situ rock.

Plate 7.6 Extensive wire mesh netting on the A82(T) at Bunbit, Loch Ness (*right*). Netting has been used in areas where shotcrete could not be applied, and is used in conjunction with rockbolting.

Plate 7.7 (*left*) Rockbolting, masonry retention and catch ditch on the A628 Stocksbridge Bypass, Sheffield. Rockbolts have also been used to retain larger blocks with a connecting metal strip.

Plate 7.8 (*below*) Extensive application of pigmented shotcrete on the A170(T) at Sutton Bank, North Yorkshire. Unfortunately, some of the areas covered with shotcrete are now beginning to act as erosion chutes.

Plate 7.9 Masonry buttressing with weepholes on the M6 at Jeffrey's Mount, Cumbria (*left*). Unfortunately, some of the drainage pipes covered by masonry buttressing have fractured. Thus water pressures have built up behind the surface, seepage has resulted, and this in turn, is leading to deterioration at the buttress – in situ rock boundary.

Plates 7.5 to 7.9 Some slope treatment measures observed in the field investigation

In active quarries, the treatment of problematic deterioration usually involves mechanical or hand scaling, re-routing of haul roads, or more commonly, simply closing off access. In disused quarries, the most common approach to problematic deterioration was to simply increase standoff distance or, more rarely, use wire mesh netting or other protective measures such as fencing. In the case of some old disused quarries, it is often unclear where ownership and responsibility lies, and if there is no legal public access, deterioration is usually just allowed to occur.

7.3.2 Controls and influences on rockslope deterioration

Due to the fact that there is such a wide range of potential influences and controls on deterioration (see Chapter Six) it was not possible to discern any widespread geographic or lithologic trends. Nevertheless, some specific trends were identified and these are considered below.

7.3.2.1 Rock mass and material properties

As expected, the field investigation revealed that deterioration in the form of block release is more pronounced where there is both a high fracture intensity and wide apertures. In rock masses which were highly fractured but where fracture aperture was very tight, slopes were much more stable. This was also the case in highly fractured rock masses where blocks were tightly interlocked. Deterioration was greater in highly fractured rock masses in weak material than in strong rock masses with an identical fracture network. The greatest deterioration occurred in slopes with both poor rock mass and material properties.

Although major discontinuity sets such as bedding planes and joints often determined the overall structure of a rock mass, it was the smaller, less persistent, often highly irregular and dense networks of fractures where most block release occurred. This is comparable to the findings of Dixon and Cox (1993) who found that on road cuttings in rhyolite and Coal Measures rocks, the *most* urgent stabilisation measures were needed in highly fractured zones, loose blocks at the top of slopes and shear zones. This was despite the fact that they identified several potential planar, wedge and toppling failures formed in association with major joint sets. These small, non-persistent and irregular fractures were the type which are formed from stress release, blasting and vegetation and weathering effects. They were commonly associated with, and formed along, small scale flaws in the rock such as laminations, lithological variations, cleavage, mineral veins, weathered bands, macro fossils and cavities.

The *type* of deterioration varied in slopes with different rock mass and material properties. In weakened materials, for example, especially those with a medium to coarse granular texture (eg sandstone, gritstone, oolitic limestone), deterioration was dominated by in situ breakdown, grain ravelling and surface scaling. Block release, whether as isolated falls, ravelling or major rockfall events, was much more common in stronger, fractured rock masses.

Deep-seated slope instability is primarily determined by rock mass properties and material characteristics are of secondary importance (Matheson 1985). However, observations made during the field investigation indicate that in dealing with deterioration, rock mass and material properties assume equal importance. For the purposes of assessing deterioration susceptibility, the four properties of discontinuity spacing, aperture, rock strength and weathering grade best describe the intrinsic nature of the rockslope independent of external environment, stress and engineering factors. These are the four properties used as key parameters in the first stage of the Rockslope Deterioration Assessment method presented in the next chapter.

7.3.2.2 Environmental, stress and engineering factors

As expected from the literature review, deterioration was enhanced where there was groundwater seepage. Sometimes this was because of in situ decomposition or disintegration associated with the presence of moisture, and other times it was because of water flow through the fracture network leading to block release. Plate 7.9 shows seepage-enhanced deterioration due to failed drainage enclosed by masonry buttressing. There was no general relationship between deterioration and slope aspect, although other climatic influences were evident. Deterioration was greater on high altitude exposed slopes than on their sheltered, low altitude equivalents. North-facing slopes, or other slopes cast permanently in shade, retained much surface water and therefore commonly had widespread cover of moss, algae and general surface staining. Material weathering in such localities was higher than slopes which received sun. There was a clear correlation between slopes with extensive groundwater seepage and the presence of vegetation. Woody vegetation in particular (eg perennial shrubs and trees), usually coincided with intensely fractured zones. This does not imply any cause and effect since it might simply be that vegetation is opportunistic, establishing where fractures and weakened material provide conduits for root growth and where moisture supply is plentiful. During periods of freezing, groundwater seepage would promote frost shattering of the rock. It could equally be the case that vegetation is the cause of the intense fracturing observed. Grasses and other low-growing, herbaceous plants were less commonly associated with intensely fractured zones and on very weathered materials, appeared to have a beneficial, binding or reinforcing effect. For vegetation to establish successfully, there is a need for the right microclimatic conditions to exist. Therefore, given the common association between localised deterioration and the presence of vegetation, there must also be an implied relationship between deterioration and climatic controls such as altitude, aspect and moisture supply.

As expected, slopes which had been excavated by bulk blasting were more fractured than those where pre-splitting had been used. However, there was no discernible difference in deterioration for slopes excavated by these two methods. This is probably because most of the small number of slopes which were pre-split were excavated relatively recently and therefore are more out of equilibrium with their environment than their older counterparts. Further, many of the pre-split slopes were cut in fissile rocks such as schist and gneiss which tended to deteriorate more. This illustrates the difficulty in attempting to establish correlation between properties for slopes where many factors interact. A much greater number of observations would be required in order for any statistical analysis to be undertaken. A similar illustration concerns the age of slopes. Older slopes, particularly those more than 100 years old, tended to be much more stable than their younger counterparts. To what extent this is a function of age and equilibrium, or of a fundamental difference in the excavation methods used is unknown.

7.3.3 Deterioration morphology

The field investigation revealed that a range of slope micro-landforms could be identified which were related to the deterioration processes acting. These were an important factor in enabling assessment of the deterioration mechanisms operating in each case. Deterioration morphology can be sub-divided into three types: (i) erosional landforms (Figure 7.1), (ii) depositional landforms (Figure 7.2), and (iii) process indicators. Some process indicators (in situ breakdown) are shown in Figure 7.3, while others are included in Figures 7.1 and 7.2.

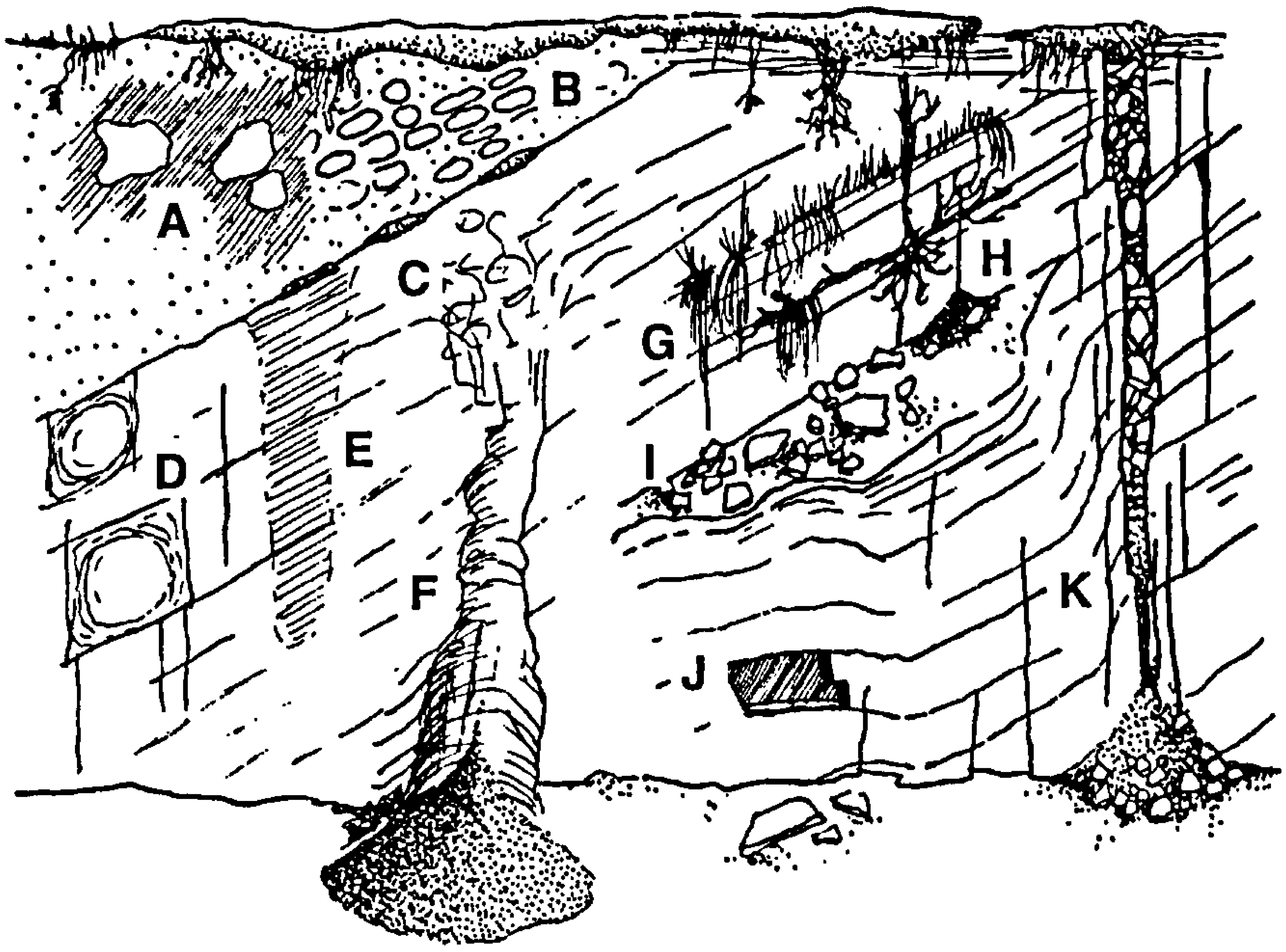
7.3.3.1 Erosional landforms

The word 'erosional' is used here in the broadest sense, to include the removal of material by artificial means as well as natural processes. Several forms could be identified.

Chutes: Chutes are quasi-channels down which loose material is transported. They are characterised by having debris piles at the foot, in some cases with a wide spread of debris in or on chute surfaces. Three types of chute were observed: *Erosional chutes* occur where rock material has been cut into by erosive agents, usually surface water runoff. They can also be formed from solution. At three slope locations, it was noted that vertical channels produced by a pneumatic impact hammer were acting as micro-chutes, down which water and fines were being transported. Similar, but larger chutes were 'created' by applying shotcrete to enlarged vertical fractures. The relatively smooth channel surface which resulted enabled a much higher velocity of surface runoff down these shotcreted 'channels' than would otherwise have been the case! These observations point to the need to consider carefully the potential for treatment measures to actually promote deterioration. *Fracture chutes* occur in fractures with a large aperture, usually due to fracture enlargement by wall breakdown. *Structural chutes* are not strictly an erosional landform. They are the product of the intersection of discontinuity planes inherent in the rock mass with the slope plane and might simply be a function of the excavation process and slope geometry. Nevertheless, they provide conduits for downslope movement of material and commonly have a build up of debris, soil and vegetation. They therefore function as chutes and are susceptible to surface erosion.

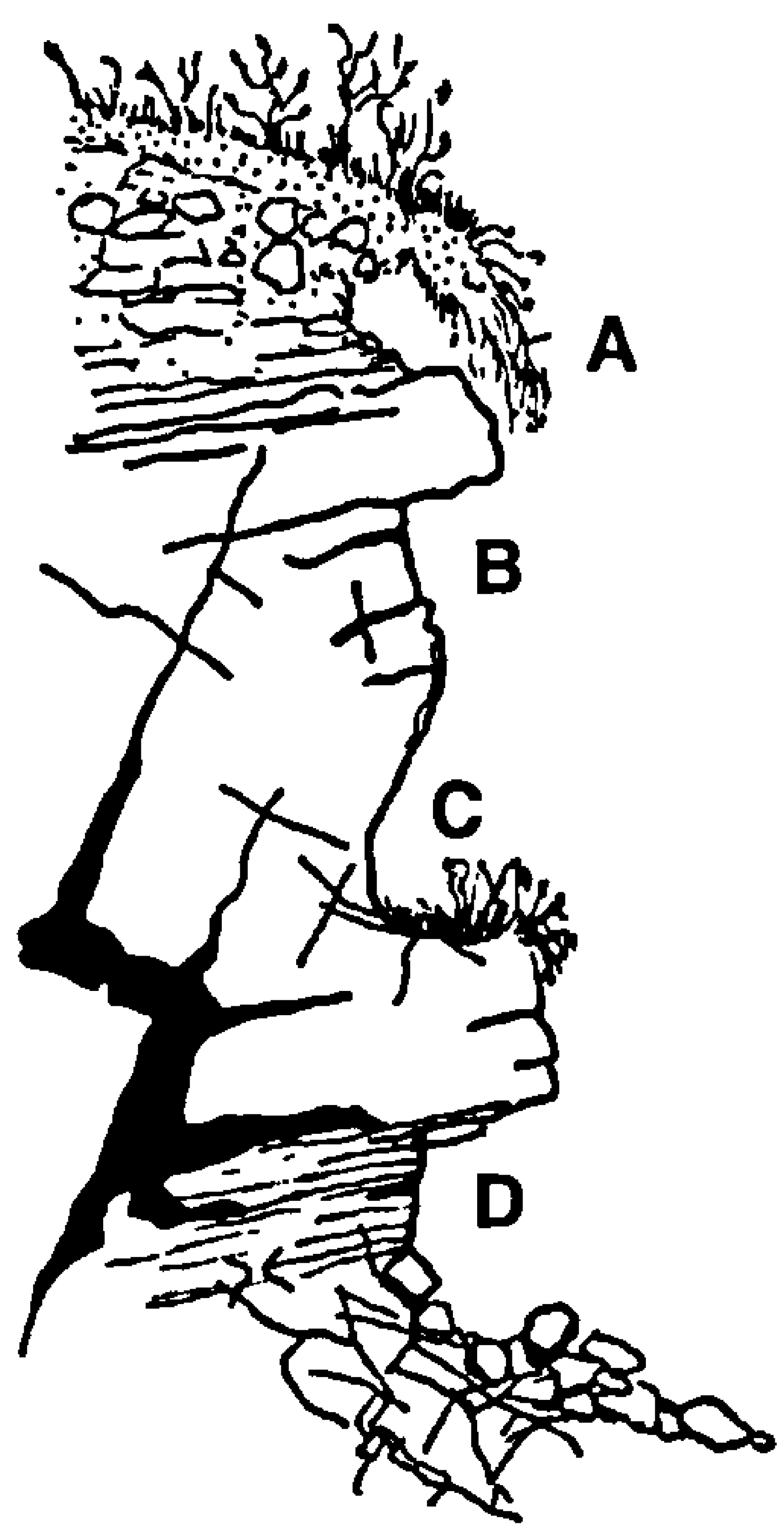
Overhangs: *Structural overhangs* can also result from the intersection of discontinuity planes with the slope plane, and might be a function of excavation procedure and slope geometry. *Erosional or composite overhangs* can occur where more competent materials are undermined by erosion of underlying weaker material or by basal undercutting in homogeneous materials. Overhangs can also result from *solution*.

Cavities: Some features can be transitional between cavities and overhangs. One example of a cavity is the alveolar structure formed from *honeycomb weathering*. Cavities of a wide range of sizes might form from *solution*. Localised and small scale cavities often form along horizontal discontinuities such as *bedding planes*, probably representing the early stages of undermining. *Man-made cavities* such as mine adits also occur.



A Surface crust and scaling
B Honeycomb structure
C Bedding plane cavities
D Onion-ring (exfoliation) weathering
E Laminations picked out by surface runoff
F Erosion chute
G Draped grass
H Root growth in cracks
I Structural chute with debris
J Surface scar
K Fracture chute

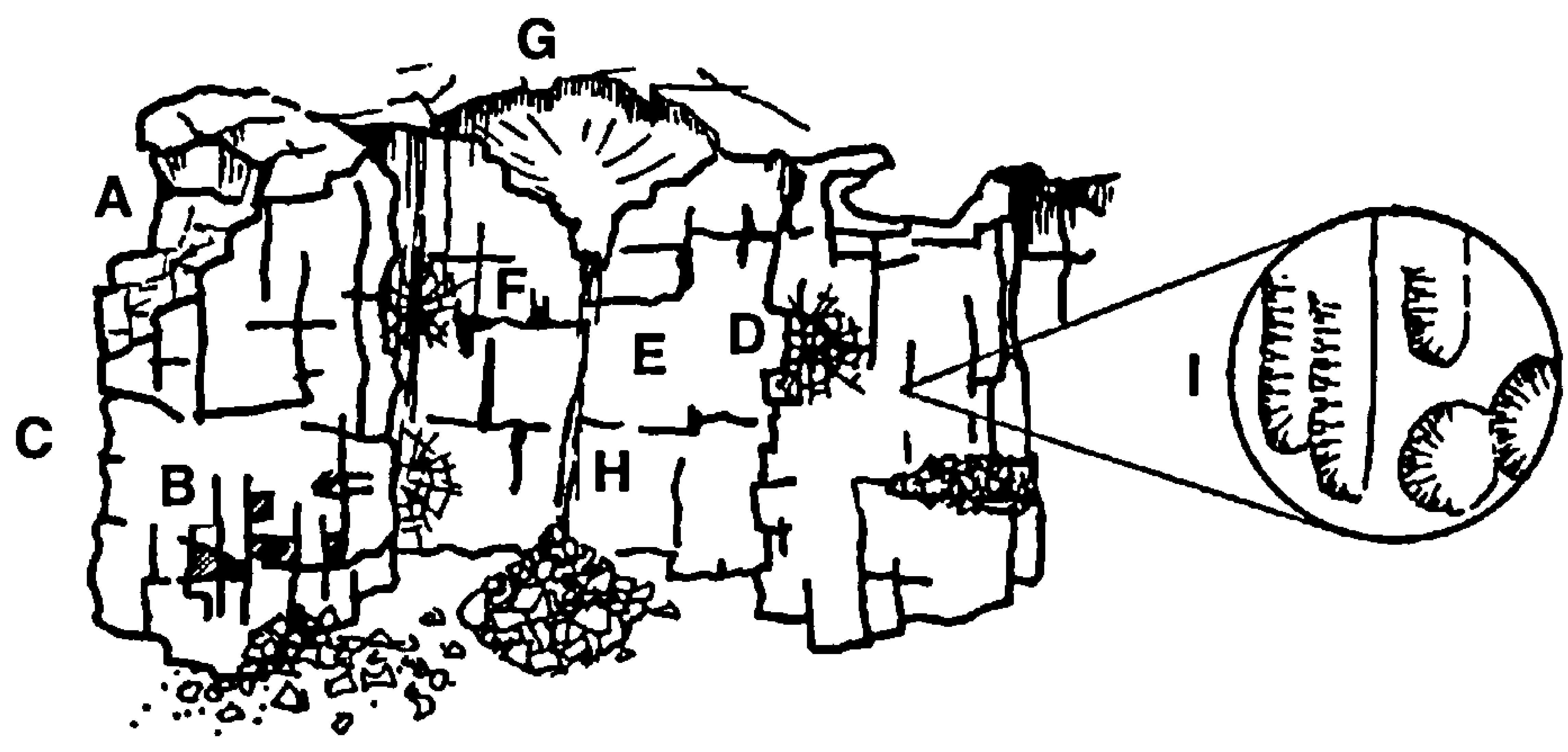
(a)



A Soil overhang
B Structural overhang
C Solutional overhang/cavity (eg tafoni)
D Erosion (composite) overhang

(b)

Figure 7.1 (a to b) Illustrations of erosional deterioration landforms



A Rockfall scar/hollow	D Blast induced fracturing	G Collapse doline
B Ravelling scars	E Headwall	H Solution chute
C Buttress	F Solution cavities	I Micro-solution runnels and pits

(c)

Figure 7.1 (c) Illustrations of erosional deterioration landforms

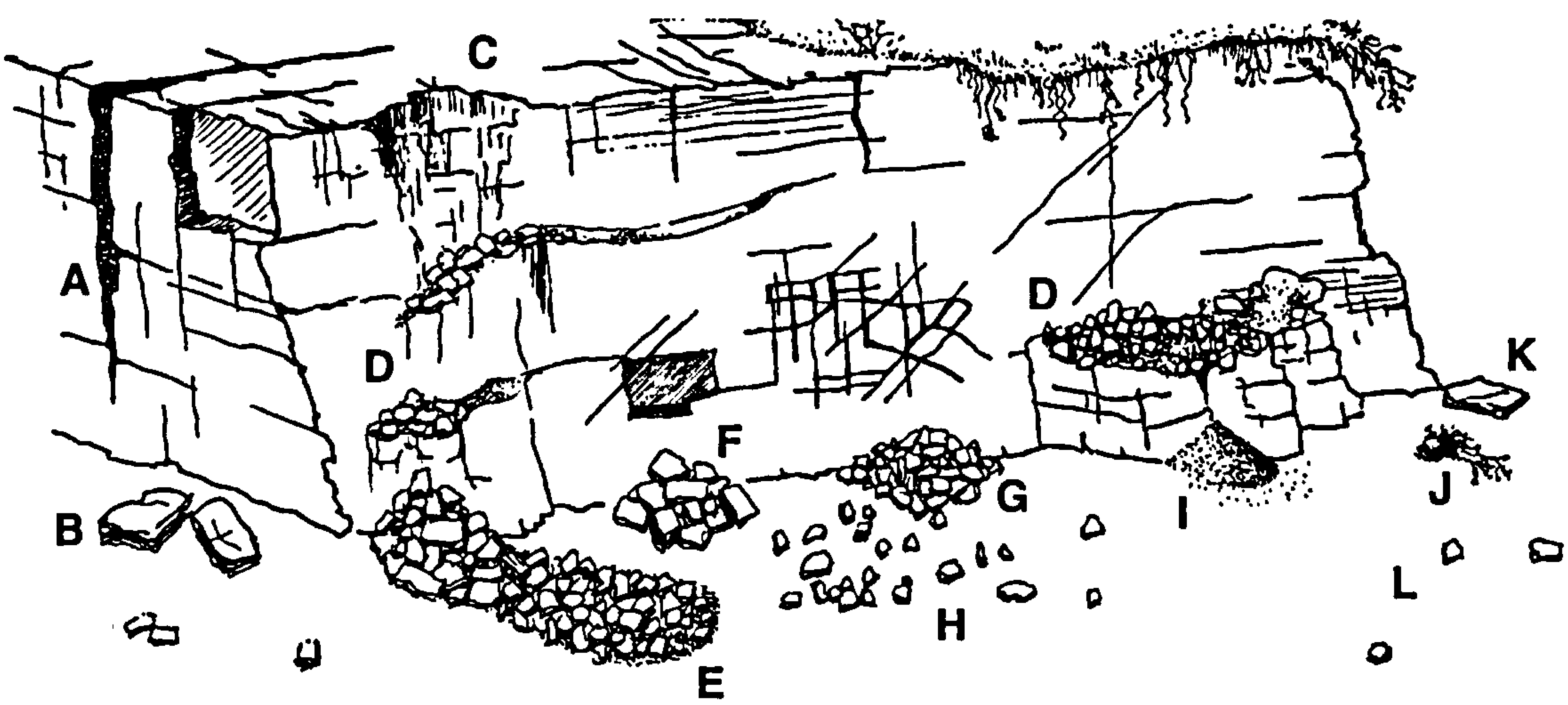
Macro landforms: Gagen (1988) reported the occurrence of a range of large scale karstic forms such as buttress and headwalls and collapse dolines occurring in disused limestone quarries in Derbyshire. Though these features were not observed in an advanced state of development in this investigation, incipient karstic forms were sometimes observed, notably in chalks and oolitic limestones. Incipient gullying was also observed in some very weak rockslopes and large scale palaeo weathering (limestone solution) features were exposed in others.

Surface scars: Scars formed when material has been removed from the slope were a very common sight in the field investigation. In some cases, scars were noticeable because the newly exposed rock was more weathered than the adjacent material, indicating that weathering had penetrated, either through the material or along discontinuities, at least to the depth of the scar. In other cases, the reverse was true, in that the scar revealed fresh, unweathered material behind. This was often an indication that the material which had been removed, was itself weathered. In yet further cases, 'scars' took the form of hollows left when a large volume of material had been removed from the slope (eg a rockfall). Scars are an excellent means of locating the likely origin of debris found at the foot of slopes.

7.3.3.2 Depositional landforms

Depositional landforms are formed from the debris which results from rockslope deterioration and can be located at the foot of the slope or on the slope itself. Depositional landforms are a useful means of estimating the likely magnitude and frequency of deterioration mechanisms involved. They can be conveniently divided into debris piles, scattered debris and isolated debris.

Debris piles: Debris piles are concentrations of debris which might be of a uniform constituent size or multiple sizes. Debris piles probably develop from one of two processes, either (i) the fall of a large volume of material in a single event, or (ii) the semi-continuous fall of material from the same location on the slope. They could of course result from activity which is transitional between these. The gradient and lateral spread of debris is related to the nature (particularly the size and shape) of the constituent material, the velocity of movement and the trajectory angle of the material as it falls. Evidence from scars on the slope suggests that debris piles formed from single fall events tend to have more lateral spread and a shallower gradient, forming quasi-fans. Gradual accumulation of debris from ravelling produces more concentrated, steeper debris piles. This is particularly true for platey fragments from shale slopes and, to a lesser extent, for sand grains.



A Fracture infilling	E Debris flow lobe	I Fines
B Isolated slabs	F Block pile	J Soil clumps
C Rockfall scar	G Stone pile	K Toppled slabs
D Debris on ledges	H Scattered debris	L Isolated debris

Figure 7.2 Composite illustration of depositional deterioration landforms

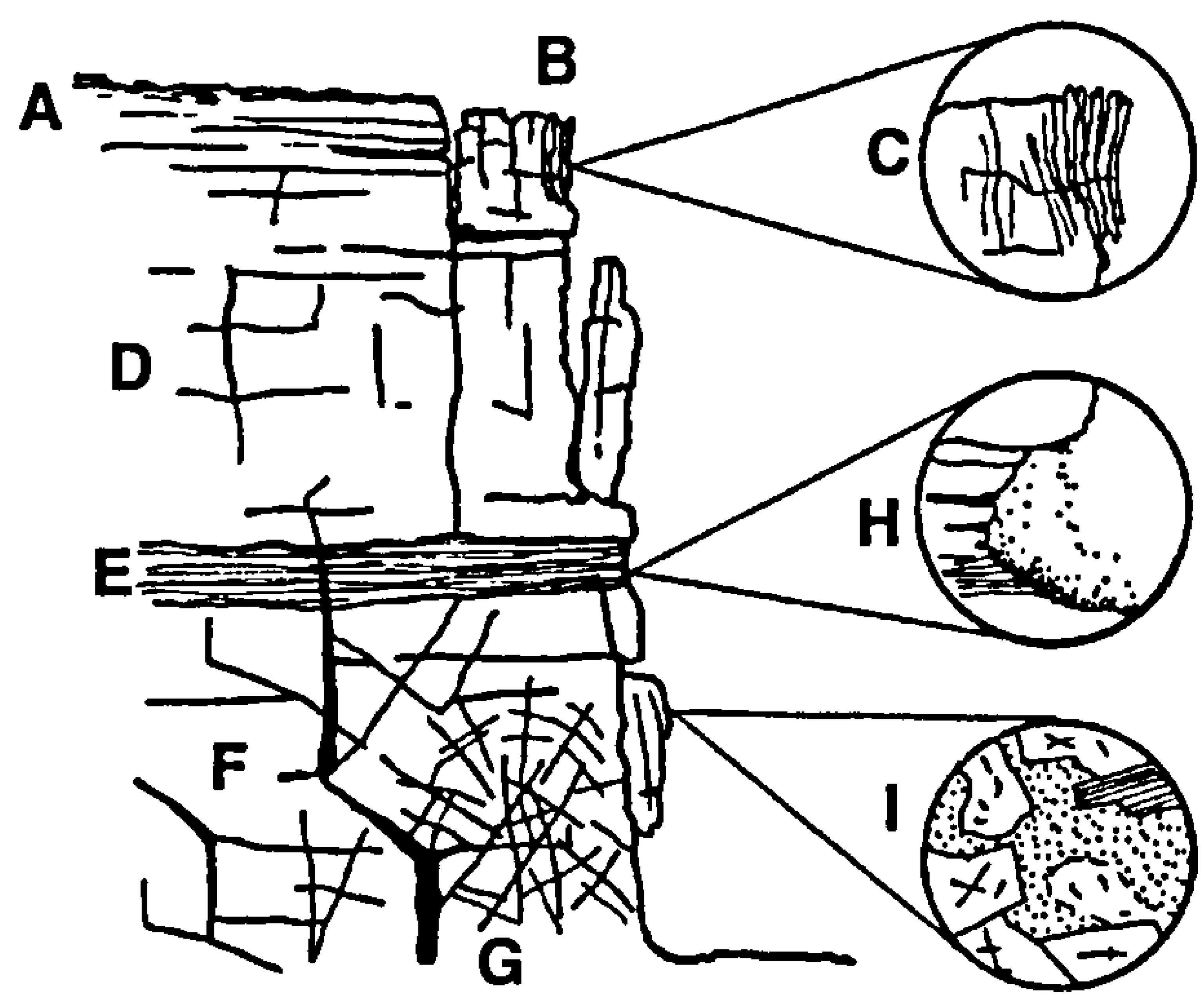
Scattered debris: Ravelling of more blocky material tends to produce an extensive scattering of material at the foot of the slope with some localised concentrations. Where these concentrations are absent, this indicates sporadic fall of material from a variety of locations and at different times. Constituent material size is often quite uniform, indicating a common control on block size.

Isolated debris: Some slopes had isolated rock fragments at the foot, indicating rare fall of material. Constituent material size was often very variable, indicating a variety of controls on block size.

Fracture infilling: A further form of depositional landform observed was the infilling of very wide aperture fractures, usually acting as fracture chutes. Infill material was highly variable and several types could be identified. These included: (i) fines resulting from the disintegration of fracture walls (effectively in situ infilling); (ii) fines washed into fractures from soil or detrital material; (iii) blocks dropped into fractures (some subsequently being involved in block wedging); (iv) mineral precipitates (eg veins, healed fractures). The occurrence and properties of these different types of discontinuity infill are considered by Welsh (1994).

7.3.3.3 Process indicators

Process indicators are features which give an indication of the cause of deterioration. These mostly relate to in situ disintegration and decomposition but the roles of surface water flow and vegetation are also worthy of special mention.



A Rebound fracturing
B Perched block
C Scaling and flaking
D Open fractures
E Weaker strata
F Solution enlarged joints
G Blast induced fracturing
H Granular disintegration
I Mineral alteration

Figure 7.3 Composite illustration of deterioration process indicators

Water flow: Since many mechanical and chemical weathering processes depend upon a supply of moisture, evidence of surface or groundwater flow is usually good indirect evidence of actual or potential weathering activity. In numerous cases where water flow was indicated there was corresponding evidence of enhanced weathering and deterioration. In dry periods, a number of indicators can be used to identify locations of regular water flow, including dampness retained in infilling materials or the rock material (evident from a colour change compared with dry rock); surface staining; penetrative discoloration; surface growth of moss and algae; preferential development of honeycomb weathering; ripples and other flow structures in accumulations of fines; the presence of vegetation; flattened or 'draped' grass; discontinuities enlarged or rounded by dissolution or water flow and individual laminae or thin beds being picked out by water erosion.

Vegetation: As indicated above, vegetation is often an indicator of waterflow. It might be associated with in situ fragmentation, particularly along, and in the vicinity of large fractures.

In situ decomposition: In a range of rocks, including sandstone, oolitic limestone, chalk and basalt, exfoliation or 'onion-skin' weathering was observed of the type that can lead ultimately to corestone development. In some cases, it was related to penetration of chemical weathering from joint boundaries, and in others, to mechanical splitting around highly contorted sedimentary slump structures. Solution was commonly observed in limestone rocks despite the short time of exposure in many cases. This had the effect of producing a range of erosional forms such as micro-solution pits, runnels and surface rounding. Honeycomb weathering was common in the Triassic sandstones. This is thought to develop because uneven case hardened surfaces developed by re-deposition of minerals at or near the rock surface are weathered differentially (Winkler 1994). The mechanism can be accelerated by salt weathering, though Mustoe (1982) favours salt weathering as the sole cause. A further form of in situ breakdown observed commonly in coarse sandstones and gritstones was the breakdown of intergranular cement such that the material could easily be crumbled. This might have been a chemical effect, a mechanical effect, or a combination of both.

In situ disintegration: In situ disintegration at the rock material scale is usually manifest as general weakening, enabling material to be crumbled easily. Increases in surface porosity might also be evident. In situ disintegration at the rock mass scale can be recognised largely from the fractures present. The increasing frequency of horizontal and sub-horizontal fractures near the surface, probably due to rebound, was a feature observed commonly, especially in massive and thickly bedded rocks. These were comparable to the thin, near-surface laminations described in section 6.3.2.3(b).These fractures tended to be regular, sub-persistent, and horizontal or sub-horizontal, often parallel to structural features such as laminae, rather than parallel to the ground surface. In contrast, stress relief fractures induced by blasting were also ubiquitous but notably irregular with a relatively wide aperture. Intensely fractured zones were evident, some probably relating to drillhole locations and others occurring in association with vegetation. On a few occasions, notably in chalk and oolitic limestone, loose blocks lying on the ground at the foot of the slope were completely shattered in situ, probably the result of freeze-thaw weathering.

7.3.4 Deterioration modes

It became evident early on in the field investigation that a wide range of deterioration modes were in operation. Determining which mode(s) was active in each case was considered to have an important bearing on assessment of the potential consequences of deterioration. As such, it was necessary identify and describe the deterioration mechanisms observed. Existing classifications of landslides were reviewed to assist in this process. However, most existing classifications either address deep-seated failures (eg Varnes 1958; Hoek 1973; Hutchinson 1988) or do not make sufficient distinction between different small scale deterioration-related processes (eg Carson and Kirkby 1972; Walton 1988). It is common practice for many of the mechanisms which result from deterioration to be described under the catch-all term of 'rockfall' (eg Figure 7.4), although authors acknowledge, either specifically or by inference, that their use of the term incorporates a variety of mechanisms. For example, in their work on rockfall hazards from excavated and natural slopes in the basement rocks of the Canadian Rockies, Franklin and Senior (1979a, 1979b) recognise ravelling, toppling, overhang collapse, ice jacking, block roll, 2D and 3D sliding, erosion and creep. This classification is the most comprehensive of those found which addresses deterioration-related mechanisms, but its wider application is limited by the narrow geographic and lithologic basis upon which it was developed. It is further limited because there is no distinction made in the classification between either event or constituent material size. Since the classification is largely based on tough gneiss, metasediment and

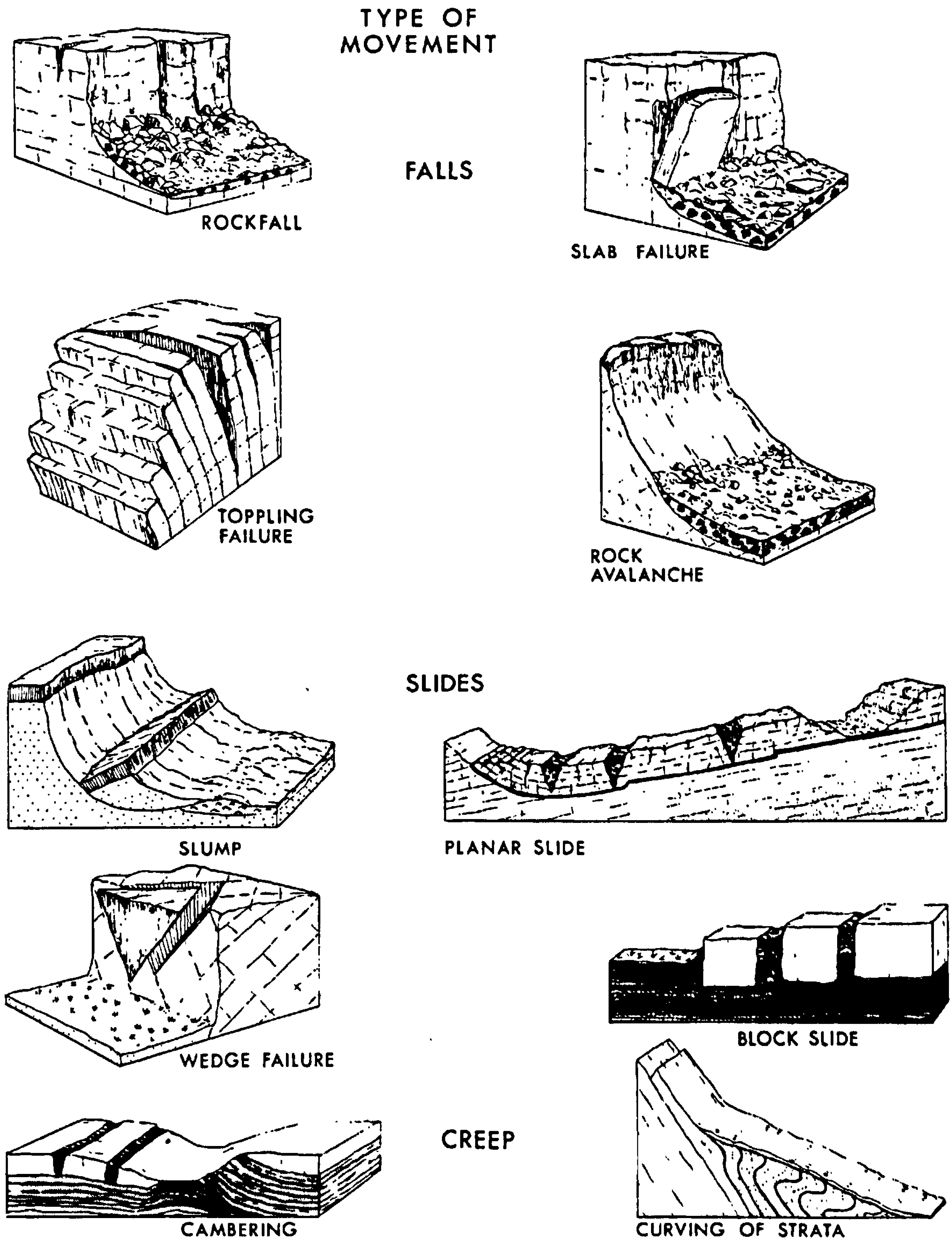


FIG. 15.15. A classification of landslides in rock.

Figure 7.4 A typical classification of mass movements (*after Selby 1993*) dolostone rocks, mechanisms which focus on material deterioration are excluded. This is despite the fact that the engineering literature contains examples of where such mechanisms are problematic (eg Williams 1990). Fookes and Sweeney (1976), in their study of stabilisation measures for degrading slopes, recognise a variety of geological features of rockslopes. These include perched blocks, flakes, slabs or pillars, overhangs, cavities, and soft, erodible beds. There is the implication that each of these might be related to different mechanisms of failure. Several authors have also classified rockfalls on the basis of constituent material size and event magnitude (Rapp 1960; Selby 1993; Whalley 1984).

A classification of deterioration modes is proposed, based on the field observations described (Figure 7.5). Categories of deterioration mode are distinguished according to frequency of occurrence, relative velocity of movement and size of constituent material. Event magnitude can also be inferred from the deterioration mode. Each mode can be recognised on the ground by the products of deterioration (eg depositional landforms) and erosional landforms (eg overhangs and scars). The classification is intended primarily for the assessment of deterioration on existing rockslopes, although it can be generally applied to proposed slopes as will be described in Chapter Eight. Each deterioration mode is distinct in terms of its maintenance and safety implications, and in terms of the remedial and protective measures suitable for its mitigation. Detailed accounts of the occurrence, hazard potential and treatment for each mode are given in Chapter Eight, but brief descriptions of each are given below.

7.3.4.1 Semi-continuous modes of deterioration

Five semi-continuous modes of deterioration are recognised: (i) *Ravelling* is the frequent and semi-continuous fall of material. Three size divisions are recognised: *grain ravelling*, relating to clay, silt, sand and fine gravel particles <20mm, *stone ravelling*, relating to coarse gravel and cobble size particles from 20 to 200mm, and *block ravelling* relating to boulder size particles >200mm. Grain ravelling can be transitional with wash erosion (see below). (ii) *Flaking* is a form of ravelling involving the frequent and semi-continuous fall of material with a distinctive platey form, as can occur in fissile rocks such as shales and slates. (iii) *Wash erosion* involves the detachment and transport of fine material entrained in surface water runoff. (iv) *Solution* involves the dissolution of soluble mineral grains and cementing material in aggressive, acid solutions, including rainwater. When this process operates at the rock mass scale and begins to affect the character of the rock mass it can be described as *karstification*. (v) *Flexural toppling* is a slow, progressive deformation and sliding of layered strata due to gravitational forces upon removal of lateral constraint.

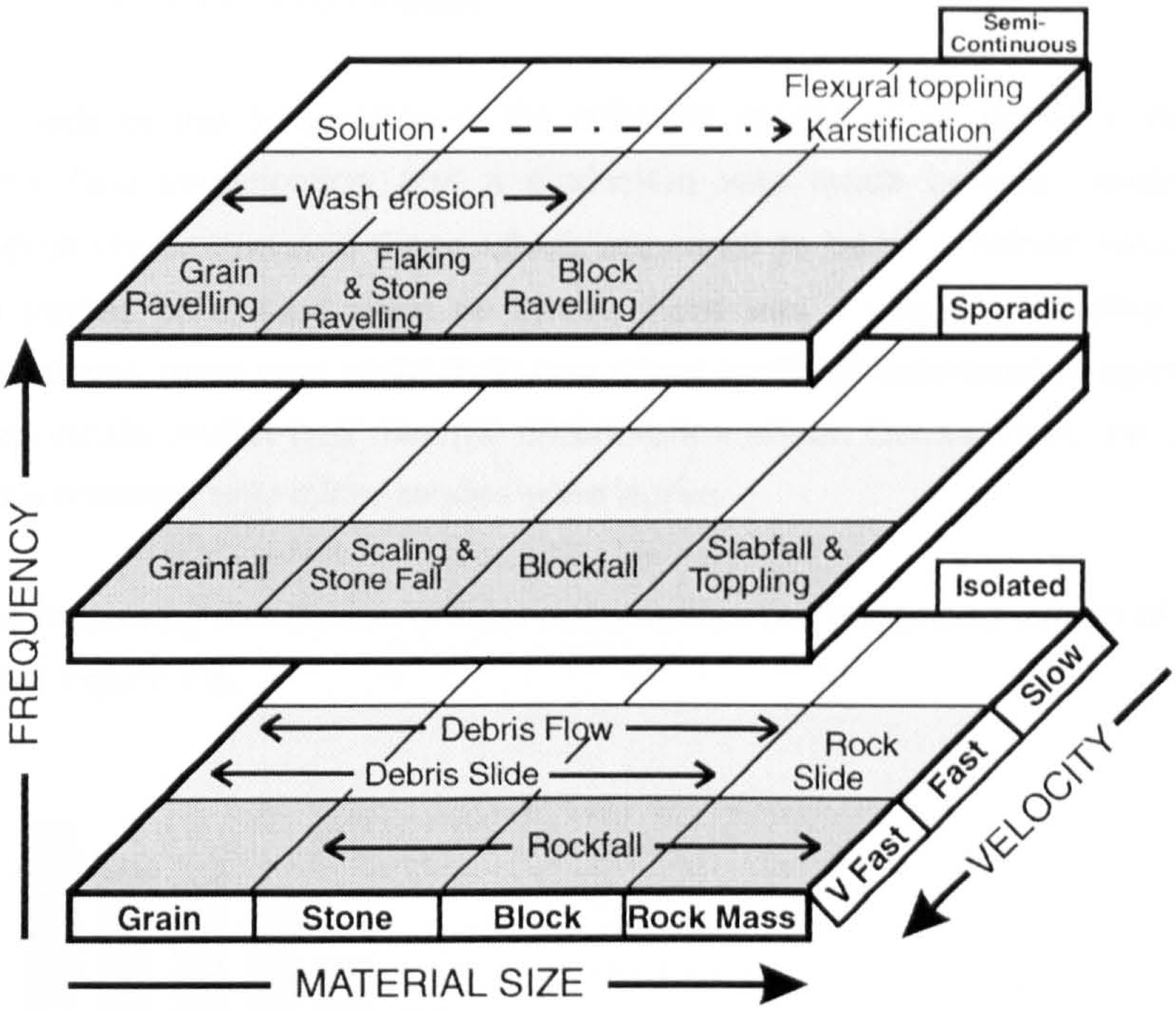


Figure 7.5 Proposed engineering classification of deterioration modes for excavated rockslopes

7.3.4.2 Sporadic modes of deterioration

Three sporadic modes of deterioration are recognised: (i) *Fall* is the occasional fall of individual fragments. The size divisions for ravelling are also recognised here to give *grainfall*, *stonefall* and *blockfall*. (ii) Contour scaling is a special form of fall involving the infrequent exfoliation of thin layers of rock material formed parallel to the slope surface. (iii) *Slabfall and toppling* involve occasional and infrequent fall of large, tabular slabs and rotation of large prismatic blocks. A typical 'a' axis dimension of such slabs and blocks is one metre. Material of smaller dimensions which falls in the same way can be regarded as stonefall or blockfall as appropriate.

7.3.4.3 Isolated modes of deterioration

Three isolated modes of deterioration are recognised: (i) *Rockfall* is used here as a specific term to describe the fall of many blocks of varying sizes in a single, identifiable event, and might involve freefall, slide, bounce and roll or a combination of these. (ii) *Debris flow* is the rapid transport of a mixture of coarse and fine particles in a partially saturated, grain-supported flow, and involves initial sliding and subsequent flow processes. (iii) *Rockslide* is the rare, large scale and rapid translational movement of rock, often along a distinct discontinuity plane. This mode is strictly outside the scope of deterioration but since smaller rockslides also occur, and since the mechanism is often largely weathering-related, it is included for completeness.

7.3.4.4 Occurrence of deterioration modes

A record was made of the occurrence of the different modes of deterioration on rockslopes examined in the field investigation and a distinction was made between **major** modes of deterioration which dominated and those which appeared to be of a **minor** nature. On many slopes several modes co-existed while on others there was a single, distinctive deterioration mode. On most slopes there was more than one minor mode of deterioration operating. These modes operated locally and/or had minimal deterioration effect. Occasionally, on slopes where deterioration was minimal, only minor modes were active.

The percentage frequency distribution of total occurrences (eg major and minor) of deterioration modes is given in Figure 7.6.

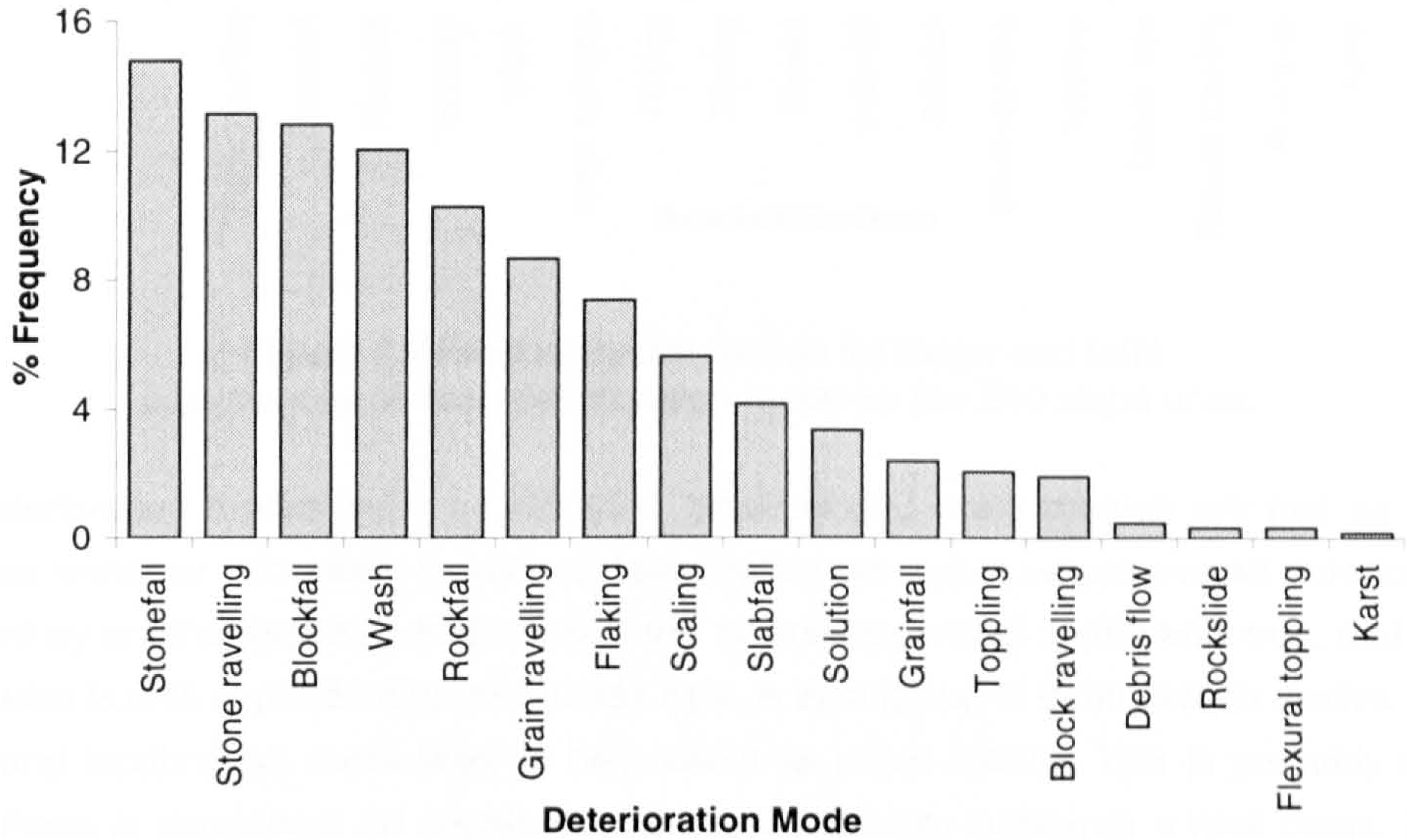


Figure 7.6 Percentage frequency distribution of total occurrences (eg major and minor) of deterioration modes

The chart shows that fall of stone-sized particles, whether as ravelling or sporadic fall, is the most common mode of deterioration. Several modes which relate to material properties are also important, including wash erosion, grain ravelling and scaling, while the large scale modes of debris flow, rockslide and flexural toppling occur least commonly. The chart presented in Figure 7.7 shows the absolute frequency distribution for both major and minor occurrences of each mode.

This indicates that some modes such as stone ravelling, grain ravelling, flaking and scaling, where they occur, are more likely to constitute the major deterioration modes. This is probably a reflection of the fact that all but the first of these modes are closely associated with particular types of rock. Flaking, for example, tends to occur in fissile rocks such as shales and slates, or metamorphosed rocks with a strong cleavage. In such rock masses, therefore, flaking is likely to dominate over other modes of deterioration. Similarly, grain ravelling and scaling are both closely associated with coarse granular rocks such as sandstone, oolitic limestone and gritstone.

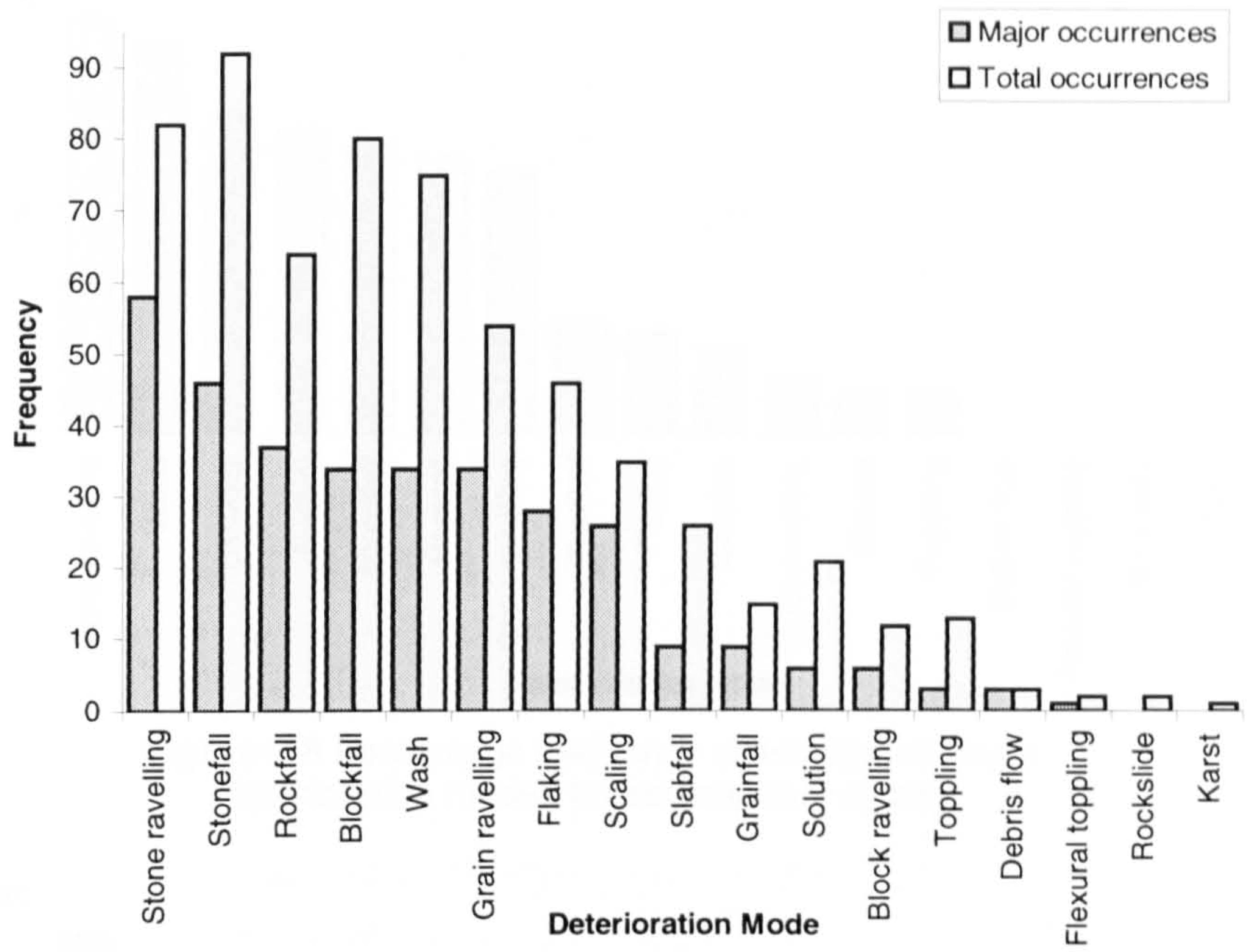


Figure 7.7 Frequency distribution for major and total occurrences of each deterioration mode for the 210 slope units.

Other deterioration modes such as stonefall, blockfall and wash erosion are just as likely to operate as minor or subsidiary modes of deterioration on slopes where overall deterioration is dominated by another process. In this case, the reverse argument is probably true, that each of these modes is less dependent on rock mass type. A third group of deterioration modes, slabfall, solution and toppling are more likely to be present as minor modes. This is probably because each of these is dependent on certain specific characteristics arising in a rock mass. Solution requires a soluble rock such as limestone, for example, and occurs whatever the other rock mass and material properties are. Slabfall and toppling are dependent upon a particular configuration of fractures and can occur in most rock masses, again, regardless of other mass and material properties.

Figures 7.8, 7.9 and 7.10 show the percentage frequency distribution of major deterioration modes for sedimentary, igneous and metamorphic slopes respectively. One of the most striking results is the *spread* of the distribution for each rock group. There would appear to be greater variability in deterioration mode for the sedimentary rocks, with increasing dominance of a few modes for the igneous and metamorphic rocks respectively. This can be illustrated by comparing the percentage frequency of stone ravelling, the most common deterioration mode for sedimentary and metamorphic rocks. In the former, the percentage frequency is 15%, but in the latter is 24%. Grain ravelling lies 7th in occurrence for sedimentary rocks with a frequency of 10%. The percentage frequency value for the 7th ranked mode for igneous and metamorphic rocks is 6% and 4% respectively, further illustrating the contrast in distributions. This contrast probably reflects the much greater range of mass and material properties represented by the sedimentary rockslopes investigated compared to the more limited range of properties for igneous and metamorphic rockslopes. Igneous rock masses, for example, are much more likely to be strong, and either relatively structureless or widely jointed. Sedimentary rockslopes, on the

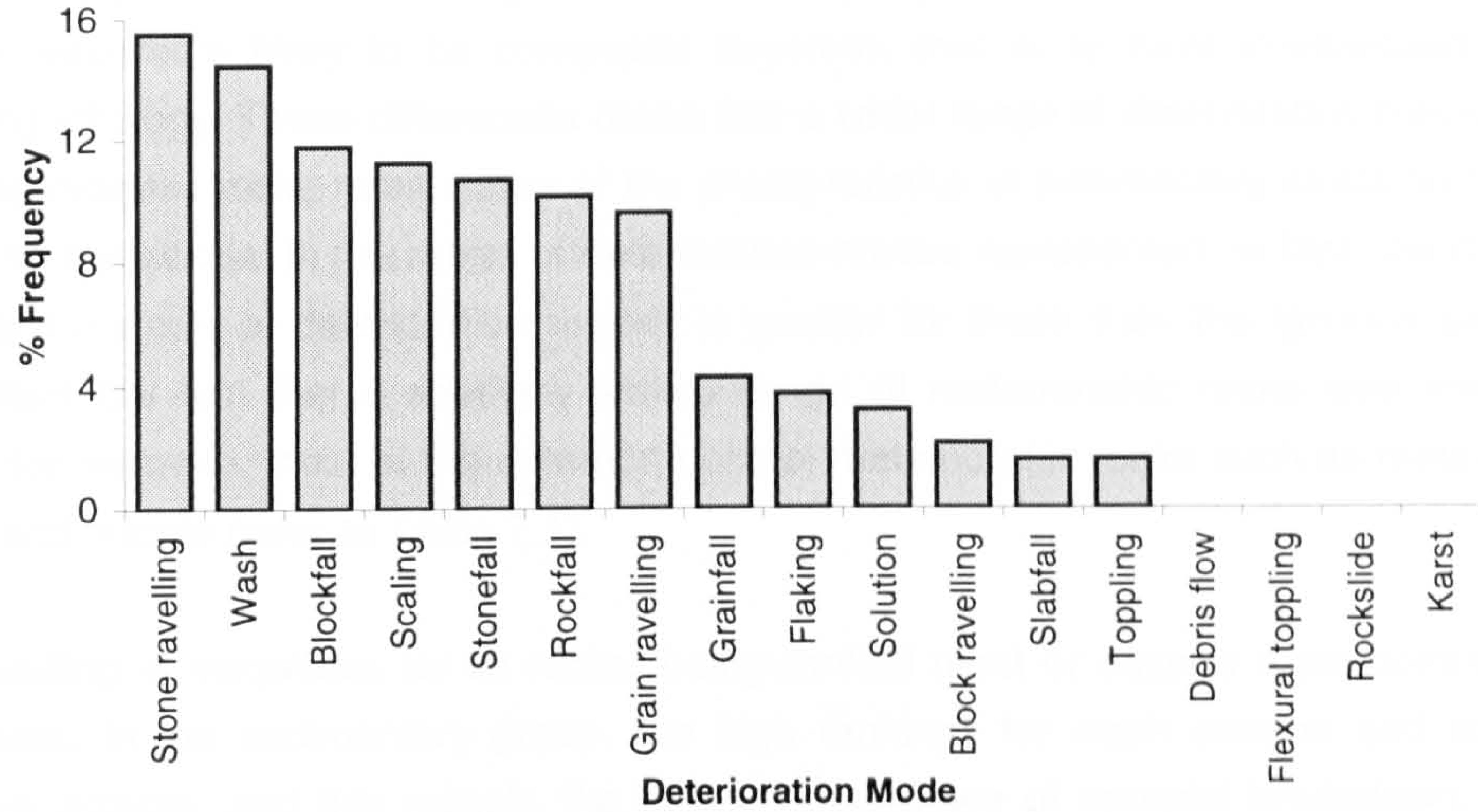


Figure 7.8 Percentage frequency distribution of major deterioration modes for sedimentary slopes

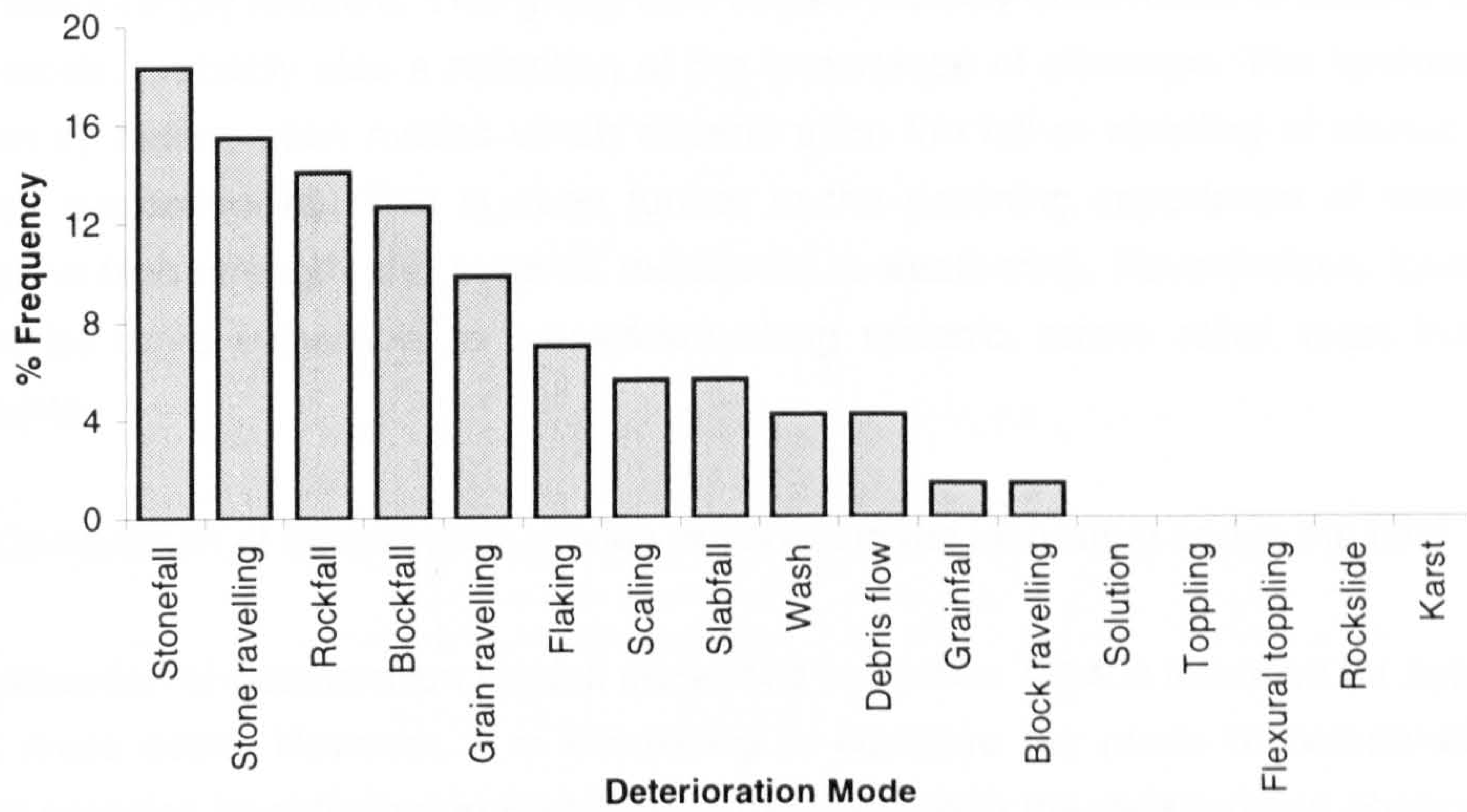


Figure 7.9 Percentage frequency distribution of major deterioration modes for igneous slopes

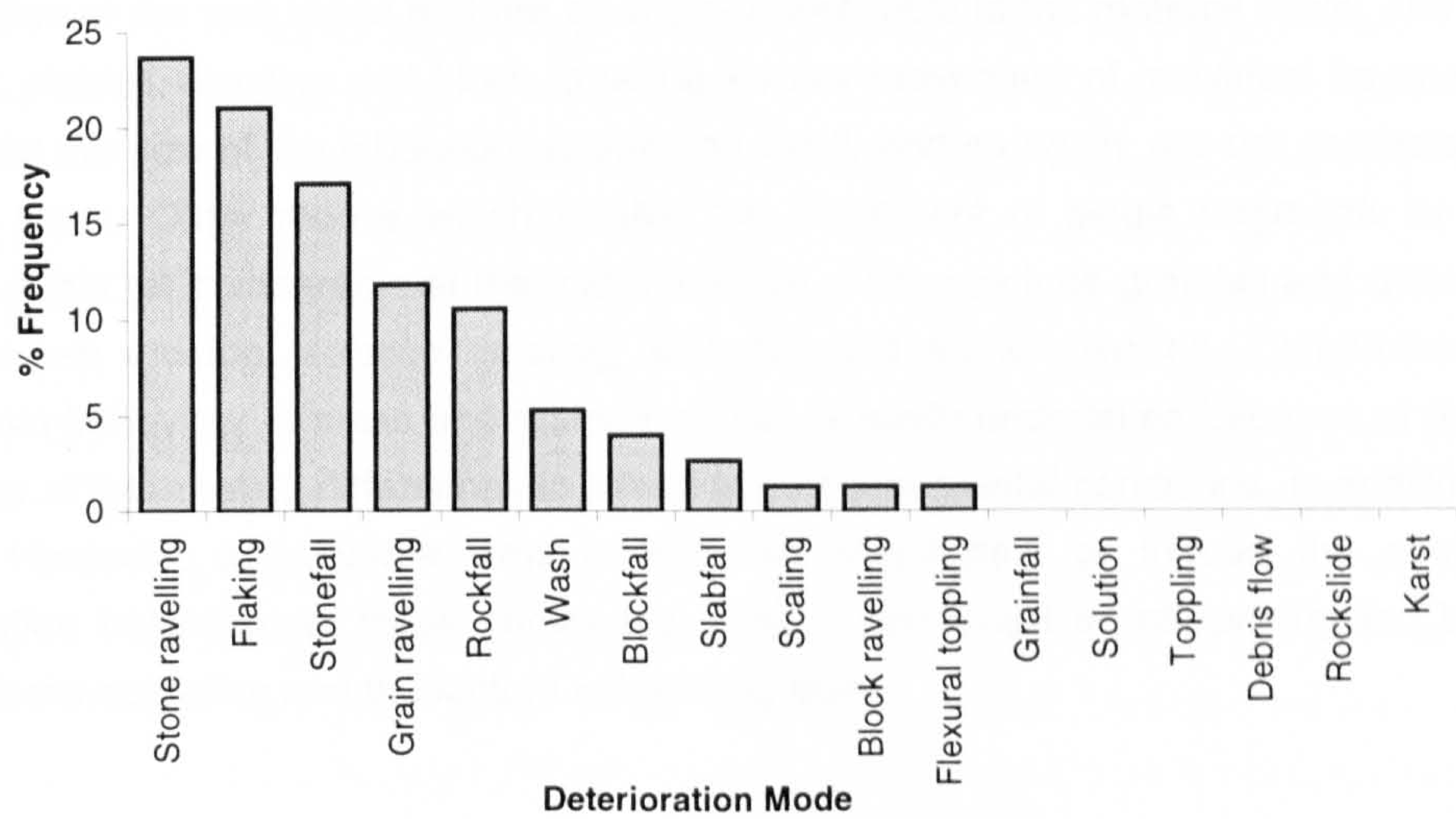


Figure 7.10 Percentage frequency distribution of major deterioration modes for metamorpic slopes

other hand, might be weak or strong, structureless, blocky, or more commonly, strongly bedded. They are also more likely to be composite (layered), that is, to have interbedded layers of contrasting lithology. These differences mean that a wider range of deterioration mechanisms is likely. Metamorphic rocks retain some of the characteristics of sedimentary strata and therefore ought to be transitional in the range of deterioration modes represented. In fact, the dominance of a limited number of deterioration modes is greater for these than the igneous group. This might reflect the fact that a relatively limited range of metamorphic rocks was investigated, omitting, for example, most of the common contact metamorphic rocks such as meta-quartzite, hornfels and marble (refer to Table 7.1).

Stone ravelling is ubiquitous for all rocks, being ranked most or second most common for all rock groups. In the sedimentary group, the high rankings for wash erosion and scaling are particularly notable, and this reflects the greater importance of material breakdown. The high ranking for flaking in the metamorphic group clearly reflects the large number of slopes which had a metamorphic cleavage, as well as a number of metasediments in which original thin bedding was strongly retained. This group also boasts the only occurrence of flexural toppling as a major mode, probably also a reflection of the importance of cleavage. The igneous group is dominated by deterioration modes which depend upon the fall or ravelling of stones or blocks defined by discontinuities. This is seen further in the declining importance of wash erosion, reflecting the high strength and material resistance to weathering. Nevertheless, igneous rocks appear to be more vulnerable to breakdown along tectonic, stress relief, blast induced and cooling joints.

7.3.4.5 Comparison of deterioration modes observed in the laboratory and in the field

The classification of deterioration modes presented in section 7.3.4 is intended for application at the rock mass scale. However, it is interesting to compare the mode of deterioration of the laboratory samples investigated in Part One of the thesis with the deterioration observed on the rockslopes from which they were obtained (Table 7.2). Some deterioration modes such as debris flow, rockslide, flexural toppling, karstification and rockfall involve movement or modification of the rock mass and are clearly not applicable at the material scale. The modes of blockfall, slabfall, toppling and block ravelling involve movement of individual fragments which far exceed the size of the laboratory specimens used, and so again, are not comparable at the material scale. Other modes which involve the movement of single fragments or grains of material might be comparable at the material scale. These include grainfall and grain ravelling, flaking, wash erosion, solution, scaling, stonefall and stone ravelling. Comparison of rock breakdown behaviour at mass and material scales is rarely undertaken because of the obvious difficulties of interpreting differences in stress and environmental conditions, in addition to scale effects. However, a converse view is that the differences, or indeed the similarities in deterioration behaviour at these contrasting scales, can assist in interpreting the breakdown mechanisms operating and the factors influencing them.

	Site number*[1]	Field deterioration mode(s)*[2]	Laboratory deterioration mode(s)*[3]
Low density chalk	63, 64	**Stone ravelling**, blockfall (*solution, stonefall, slabfall, rockfall*)	Disintegration, intense fracturing, shallow scaling and multiple flaking
Magnesian limestone	1	**Grain ravelling**, **wash erosion**	Fracturing, scaling, minor fragmentation and cavity development
Oolitic limestone	57, 58, 59	**Scaling**, blockfall, rockfall, stonefall, wash erosion, stone ravelling, grain ravelling (*block ravelling, flaking*)	Disintegration, fracturing, scaling and granular loss
High density chalk	72, 73, 74	**Stone ravelling**, rockfall (*grain ravelling, flaking*)	Fracturing, deep scaling and breakage
Sparry limestone	40	**Stonefall** (*stone ravelling, wash erosion, karstification*)	Minor fracturing
Weathered sandstone	25	**Grain ravelling**, grainfall, stone ravelling, stonefall (*blockfall, slabfall, rockfall*)	Granular loss, minor fracturing and fragmentation
Calcareous sandstone	100, 101, 102, 103	**Stonefall**, **wash erosion**, **stone ravelling**, grain ravelling, blockfall (*solution, toppling*)	Fracturing, scaling and minor granular loss
Micaceous sandstone	22	(*Stonefall, blockfall, scaling*)	Granular loss, minor fracturing, scaling and fragmentation
Laminated siltstone	93	Stone ravelling, block ravelling, grain ravelling (*stonefall, blockfall, rockfall*)	Severe disintegration and intense fracturing, multiple flaking and breakage
Metasediment	209, 210	**Flaking**, stonefall, grain ravelling, stone ravelling (*wash erosion, blockfall*)	Minor fracturing

Table 7.2 Comparison of laboratory and field deterioration modes

Note 1 Site number as given in Appendix 9.A
Note 2 Field deterioration modes as given in Appendix 9.Ac. Formatting indicates relative importance (bold and underlined indicates a major mode of deterioration, underlined indicates a major mode co-existing with others, no formatting indicates transitional between major and minor mode, and italicised and in parenthesis indicates a minor mode).
Note 3 Laboratory deterioration modes as given in Table 5.1

The data shown in Table 7.2 above indicate that for some rock types there is considerable similarity in deterioration mode at both field and laboratory scales. This is the case with the weathered sandstone, for instance, selected from Coal Measures gritstone deposits at Blubberhouses road cutting on the A59 in West Yorkshire. The cutting is subject to two very distinctive forms of deterioration: (i) granular disintegration leading to grainfall, grain and stone ravelling, and (ii) the movement of large blocks and slabs formed by major joint sets. Breakdown of the laboratory samples by granular loss and minor fragmentation is very similar to the deterioration modes operating at the material scale in the field. Similarity can also be seen between field deterioration of both the low and high density chalks which was dominated by breakage and fragmentation along fractures and the style of disintegration which occurred due to experimental weathering. Scaling also features as a major deterioration mode at both field and laboratory scales for the oolitic limestone. The sparry limestone, micaceous sandstone and metasediment show similarity in their general resistance to deterioration in both the laboratory and the field. The only rock which shows a strong contrast between field and laboratory deterioration mode is the laminated siltstone which was obtained from a working open cast coal operation. Deterioration of laboratory specimens was dominated by very closely spaced fracturing and multiple flaking along pre-existing laminations. In the field, breakdown involved the fall and ravelling of stone and block sized fragments, formed along bedding planes and irregular blast-induced fractures. The contrast is probably due to the fact that in the field, the rock mass had only recently been exposed (a matter of days or weeks) and so weathering had little time to take effect. It is highly likely that with continuing exposure to the environment, the rockslope would begin to exhibit similar signs of breakdown along laminations.

7.3.5 Rock mass types

In the course of the field investigation it became apparent that (i) deterioration modes were strongly related to three rock mass properties, namely rock type, fracture network and rock strength, and that (ii) distinctive rock mass types could be identified on this basis.

7.3.5.1 Existing classifications of rock mass

Several classifications of rock masses have been developed (eg Duncan and Goodman 1968; Aydan and Kawamoto 1990; CIRIA/CUR 1991, given in Figure 7.11; Aydan et al 1992; Clerici 1992; Hoek 1994; Goodman 1995) and these are briefly described for context.

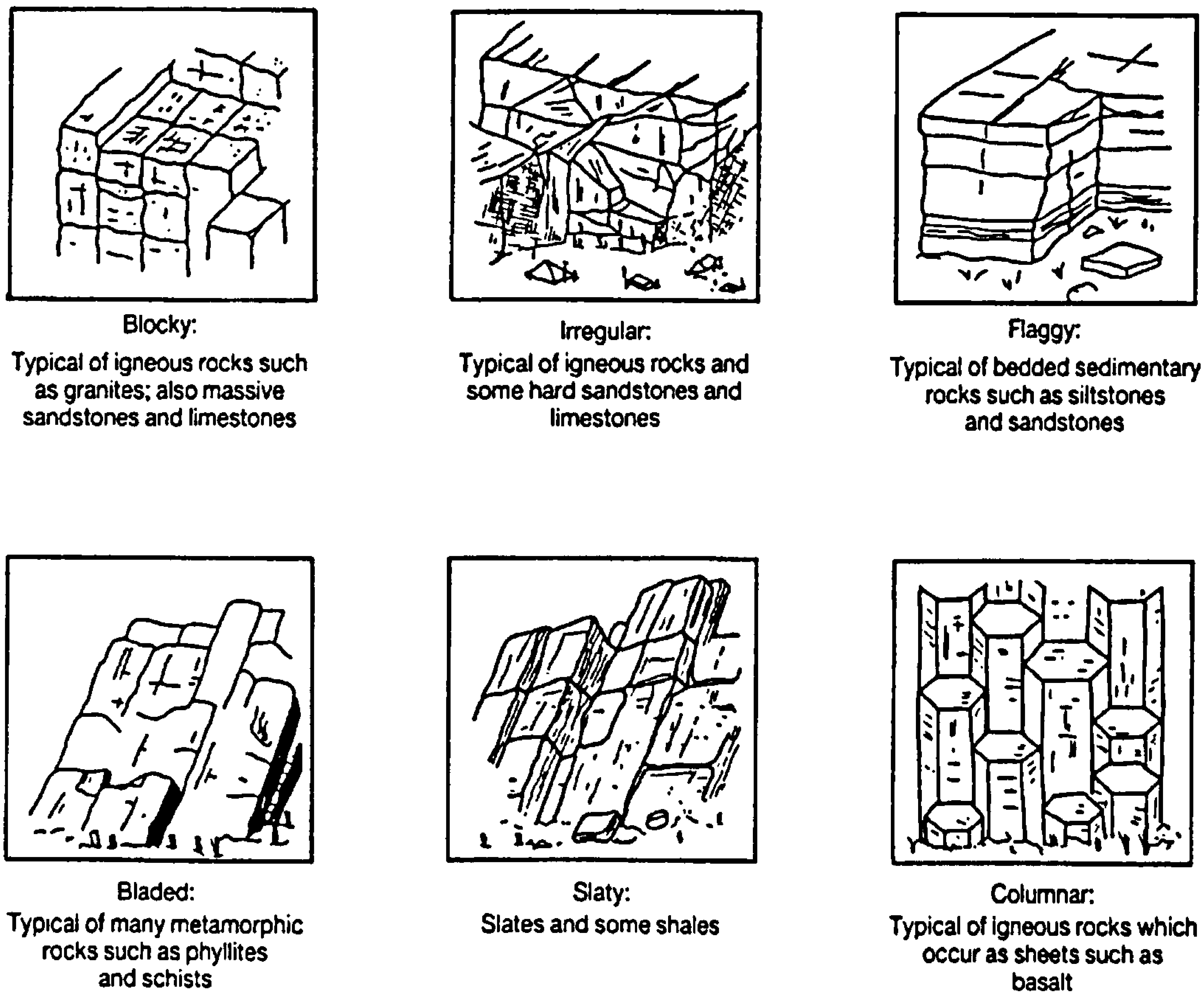


Figure 7.11 Idealisations of rock mass structure (*after CIRIA/CUR 1991*)

The scheme developed by Clerici (1992) applies for general engineering geology purposes and is only for weak rocks. Rock masses are classified on the basis of structure, rock texture, strength, and joint spacing. The schemes developed by Hoek (1994) and Aydan and Kawamoto (1992) are based on joint pattern and intensity. The former is aimed at understanding rock mass strength while the latter was specifically aimed at structure modelling of rock masses. Goodman (1995) developed a classification on the basis of joint characteristics (eg number of sets, arrangement, aperture, infilling); lithology (eg porosity, clay minerals, solubility, weatherability, fissility); and rock mass structure (eg foliated, cavernous, corestones). The schemes developed by Duncan and Goodman (1968) and Aydan et al (1992) are perhaps the most applicable to this work since they attempt to model different types of rockslope failure mechanism in terms of rock mass characteristics. In the case of the Duncan and Goodman (1968) scheme, for example, a wide range of properties are taken into consideration including rock structure, failure mechanisms, rock strength, and joint continuity. These are useful classifications but are difficult to apply directly to evaluation of rockslope deterioration because the categories are defined with respect to the potential for instability which is dependent upon major joint sets. Also, they do not specifically address the role played by non-persistent discontinuities such as those induced by stress relief, blasting and weathering, which are often the focus of deterioration. Nevertheless, there are some useful broad concepts which can be incorporated into a classification designed to model rock mass types relevant to deterioration-related failure.

7.3.5.2 A proposed classification of rock masses

A classification of rock mass types is proposed, specifically designed for use in assessing rockslope deterioration. The primary distinguishing factors for each rock mass are the arrangement of fractures and the rock mass structure. Three primary categories are recognised: Massive, Layered and Blocky, each having several sub-types. Massive rock masses are sub-divided into weak and strong massive; layered rock masses are sub-divided into layered, fissile and composite; and blocky rock masses are sub-divided into regular and irregular blocky. These descriptors refer to the pattern of fractures, not closed discontinuities. For example, a rock mass which contained well defined, horizontal bedding but where boundaries between strata were closed, would be described as massive, not layered. Additional descriptive terms can be used as appropriate. For example, layered rock masses can be described as vertically layered or prismatic, blocky rock masses can also be described as prismatic or rubbly. Three subsidiary rock mass types are also recognised which occur within any of the primary rock mass types. These are (i) intensely fractured zones; (ii) soluble rock masses; and (iii) composite rock masses. Brief definitions of these are given in Figure 7.12, but fuller descriptions, together with details of occurrence and geotechnical implications are given in Chapter Eight.

7.3.5.3 Occurrence of rock mass types

The frequency distribution of rock mass types observed in the field investigation is given in Figure 7.13. The chart shows that layered and blocky rock masses are significantly more common than other types. Since this is, in part, a reflection of the range of rockslopes investigated, it is helpful to consider the distribution of rock mass types with reference to the three rock groups, sedimentary, igneous and metamorphic (Figures 7.14, 7.15 and 7.16). These charts show that some rock mass types are absent from some rock groups. Weak massive rock masses notably, only occur in the sedimentary group, and fissile and composite rock masses are absent from the igneous group.

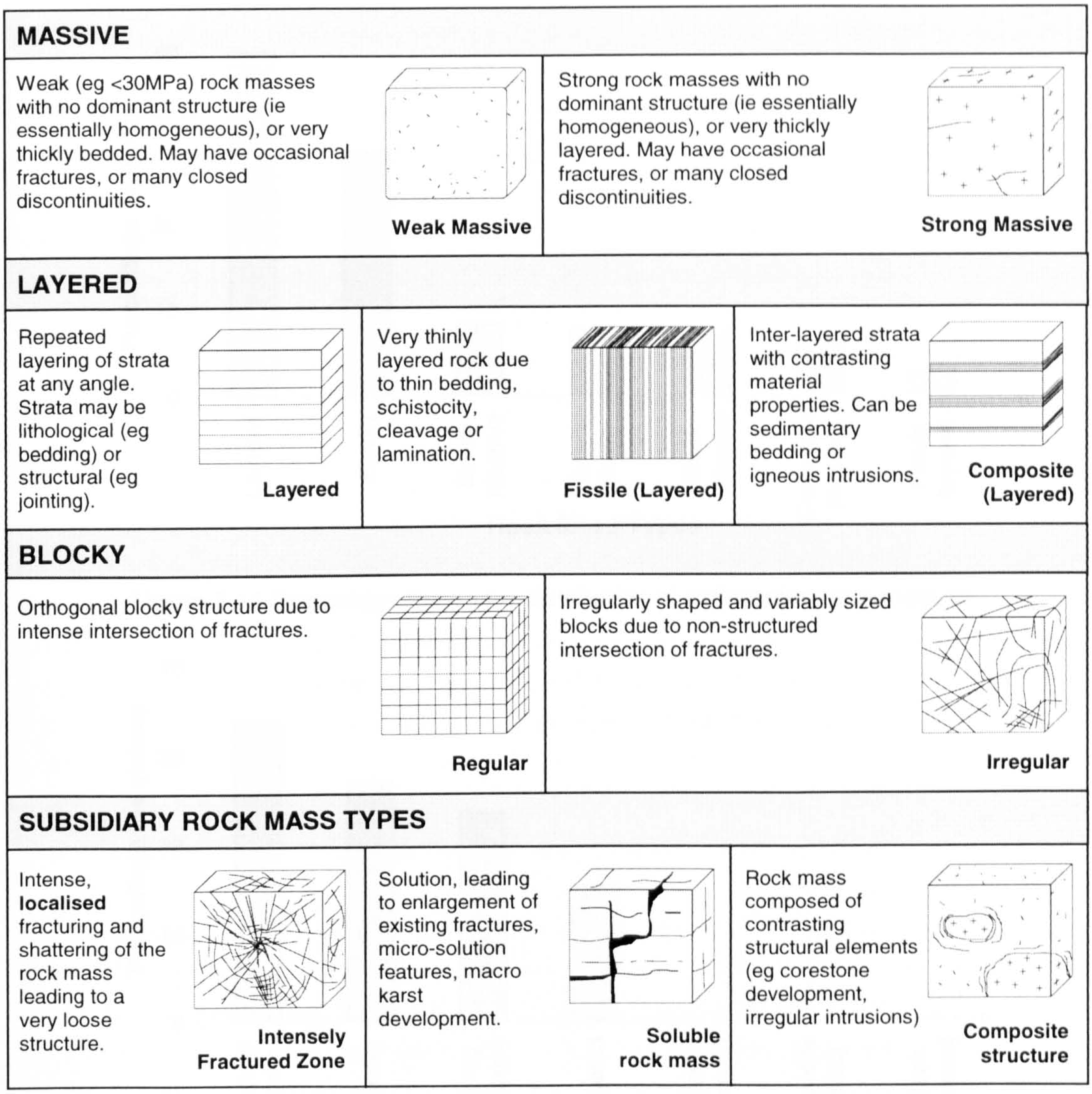


Figure 7.12 Classification of rock masses for evaluation of rockslope deterioration

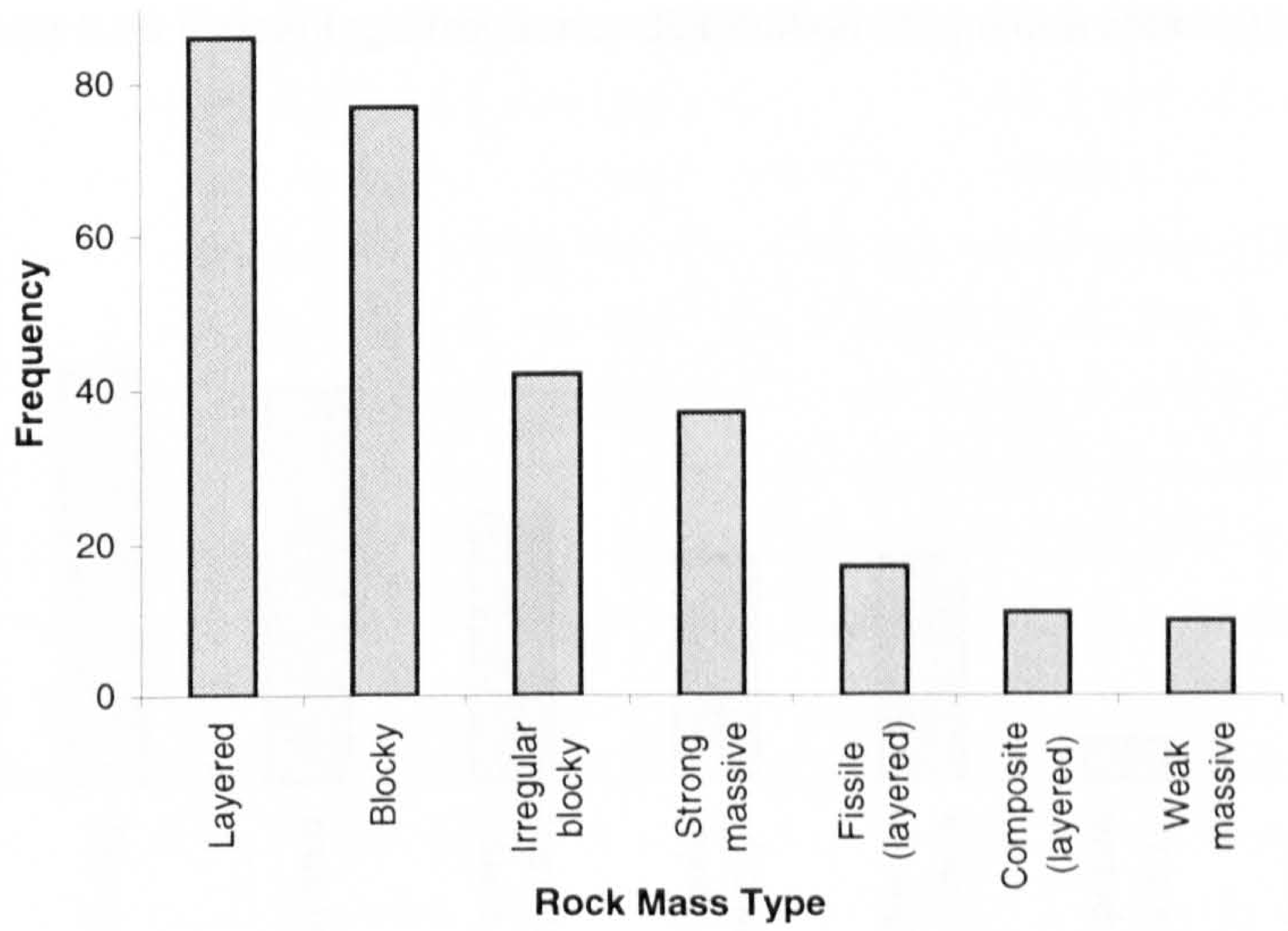


Figure 7.13 Frequency distribution of rock mass types

(The total number of slope units was 210, but in some cases more than one rock mass type co-existed)

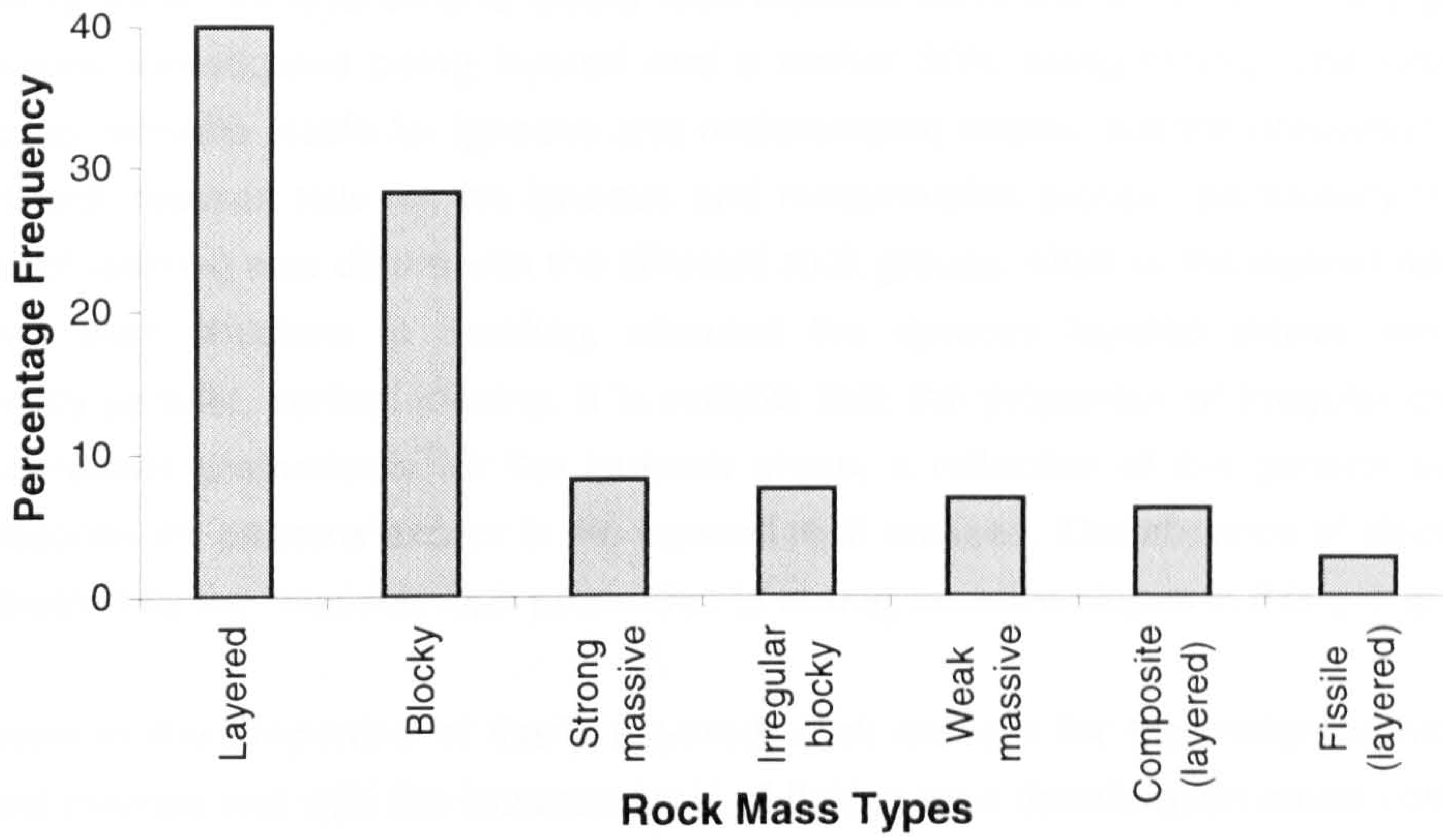


Figure 7.14 Percentage frequency distribution of sedimentary rock masses

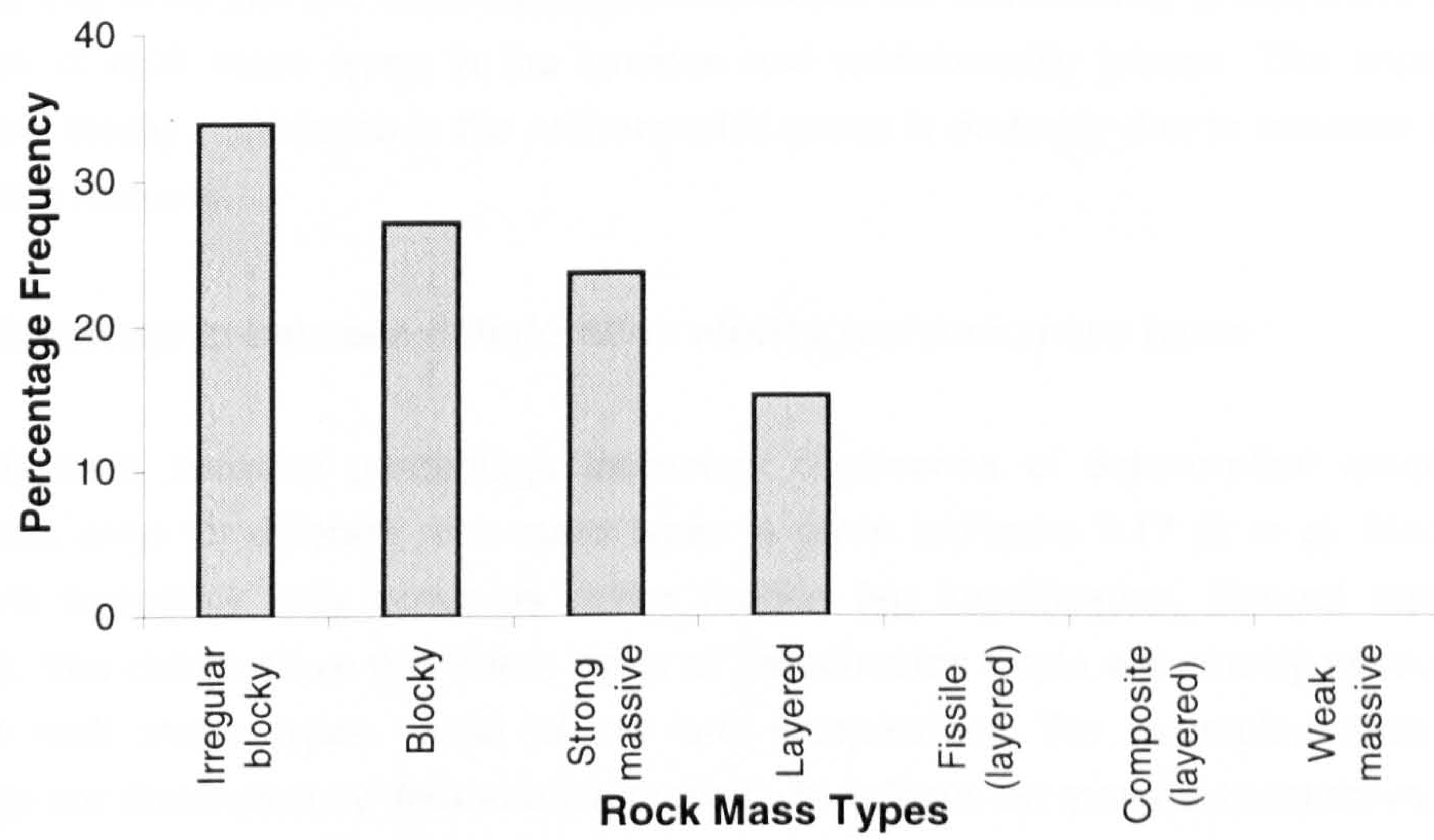


Figure 7.15 Percentage frequency distribution of igneous rock masses

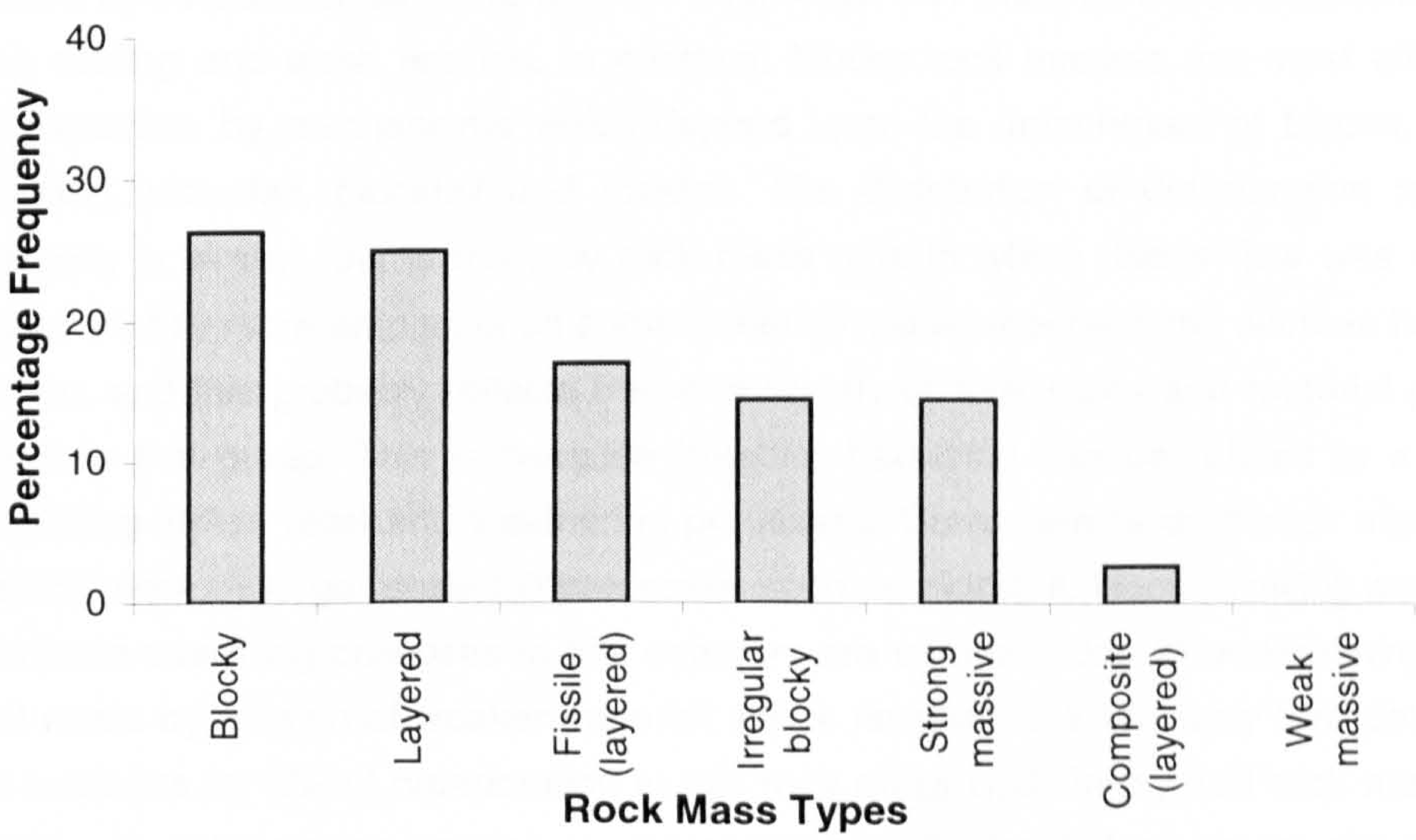


Figure 7.16 Percentage frequency distribution of metamorphic rock masses

These charts show that layered and blocky rock masses dominate the sedimentary group, with 40% of slopes investigated being layered and a further 25% being blocky. The proportion of blocky slopes remains stable for igneous and metamorphic slopes, but the proportion of slopes in layered rock masses falls for the igneous and metamorphic groups, particularly the former. The *cause* of layering also differs with the different rock groups. Most of the layered sedimentary slopes owe their structure to bedding, whereas the igneous layered slopes were usually dominated by parallel, vertical jointing. It is notable that the proportion of irregular blocky rock masses increases dramatically for the igneous group, a reflection of the general absence of regular discontinuity patterns except in the layered rock masses. The absence of discontinuities is also indicated by the relatively high proportion of strong massive slopes in this group.

The increase in the proportion of fissile (layered) rock masses for the metamorphic group is notable and marries well with the increased role of flaking as a deterioration mode noted earlier. The absence of any weak massive rockslopes in the igneous and metamorphic groups reflects the relatively high rock strength for these compared with sedimentary slopes. Overall, it is interesting that while just two rock mass types dominate the sedimentary group, there is broader distribution of rock mass types in the igneous and metamorphic groups. The importance of layered and blocky rockslopes in the metamorphic group is probably due to retention of original sedimentary features.

7.3.6 Relationship between deterioration modes and rock mass types

The relationship between percentage frequency occurrence of deterioration modes (major occurrences only) for different rock mass types is given in Figure 7.17 (a to g). Modes which show zero frequency only occur as minor modes (eg karstification, flexural toppling and rockslide). The charts show that some types of deterioration mode are closely associated with particular rock mass types, while others are independent. For example, weak massive rockslopes are dominated by deterioration modes which focus on material breakdown, including grainfall, grain ravelling, wash erosion and contour scaling. Fissile rock masses are strongly dominated by the flaking mechanism as mentioned previously. Strong massive rockslopes are most affected by occasional fall of stone sized fragments, and material-based mechanisms such as surface scaling and wash erosion. In contrast, blocky rock masses are most affected, as would be expected, by mechanisms which depend upon the detachment of blocks, including stone ravelling, stonefall, blockfall and rockfall. The distribution of deterioration modes for irregular blocky is similar, but is the only rock mass type in which debris flow was observed. There is also slightly more emphasis on some material-related mechanisms such as flaking and wash erosion, and this probably reflects the wide variety of rock mass and material properties encountered in this group. This is because irregular fracturing can be related to a range of causes including stress relief and weathering processes. Composite layered rock masses tend to be affected more by larger scale fall processes such as blockfall, block ravelling and rockfall. This is because overhang collapses in this category are common due to undermining of more competent strata by erosion of weaker material. Block ravelling is a relatively rare deterioration mode but accounts for 8% of deterioration in this rock mass type. In layered rock masses, the widest range of deterioration modes is represented, with the fall of stone sized material dominating slightly. Charts showing the absolute frequency distribution (major and minor

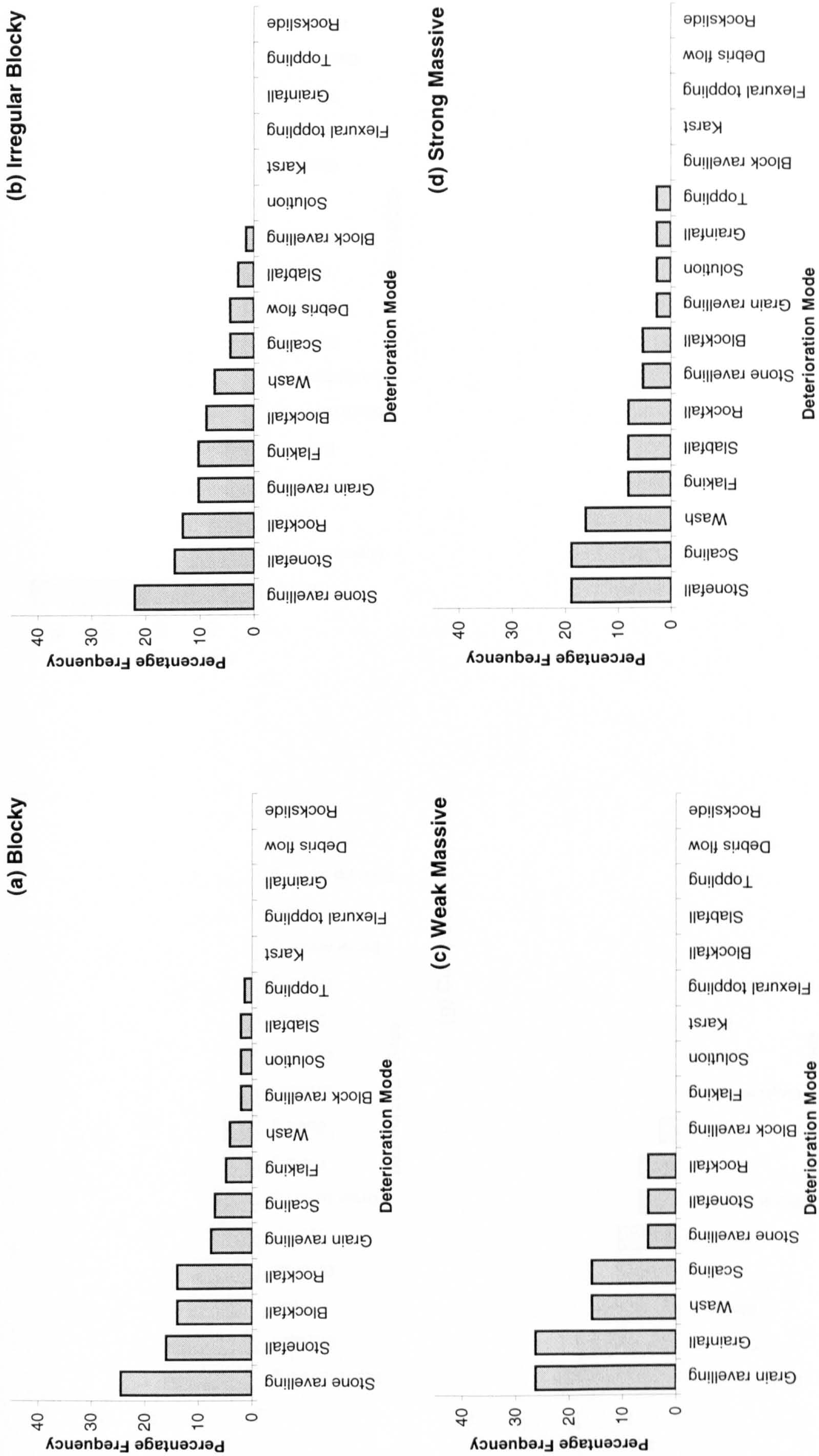


Figure 7.17 (a to d) Percentage frequency distribution of deterioration modes for each rock mass type

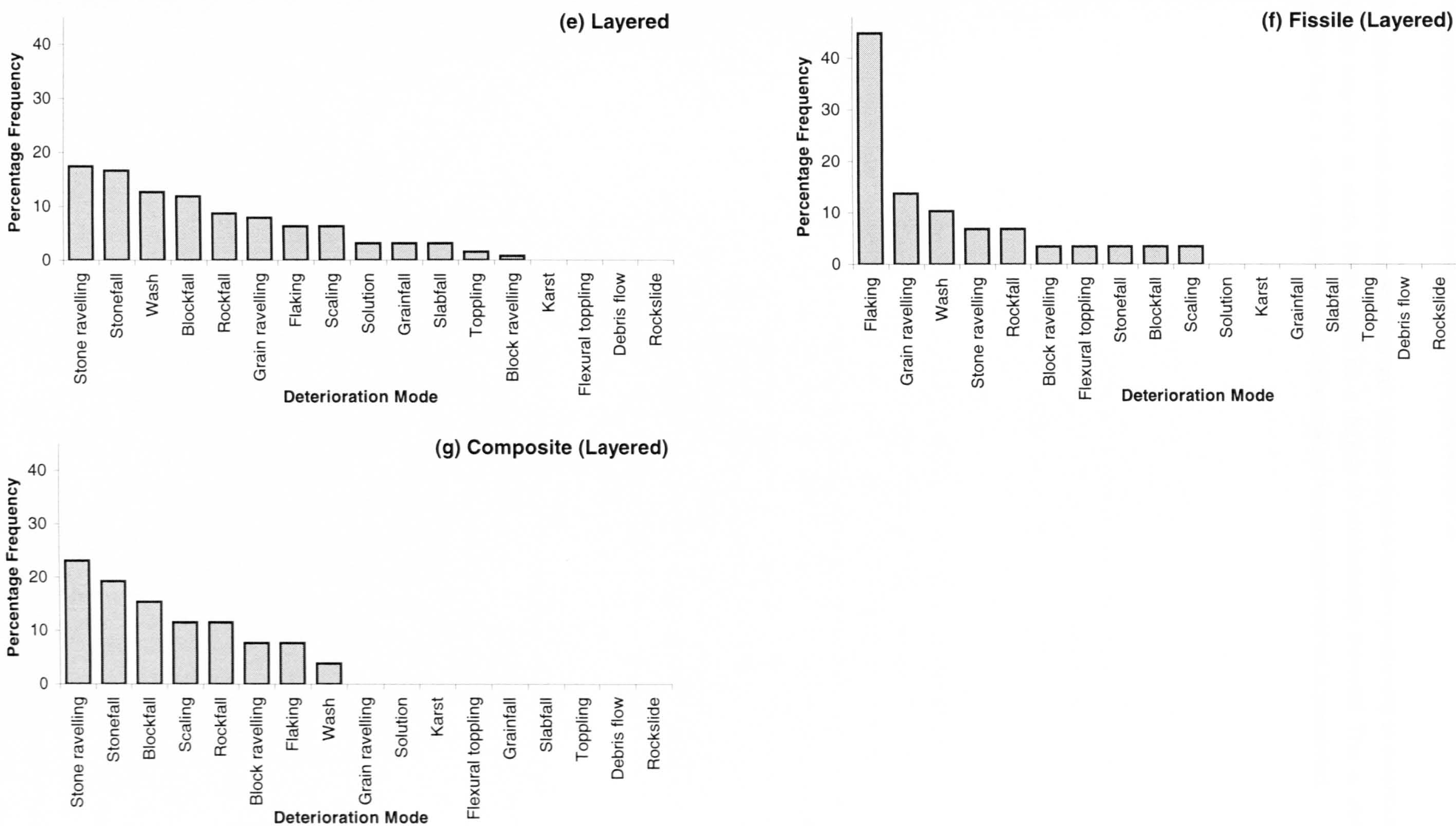


Figure 7.17 (e to g) Percentage frequency distribution of deterioration modes for each rock mass type

occurrences) of deterioration modes for each rock mass, for sedimentary, igneous and metamorphic groups and overall, are given in Appendix 7.A.

The data described above suggest that rock masses have a built-in propensity to deteriorate in a certain way and as such, there might be a degree of predictability involved. This is utilised in Chapter Eight, in which the Rockslope Deterioration Assessment method is presented.

CHAPTER EIGHT
ROCKSLOPE DETERIORATION ASSESSMENT (RDA)

8.1 Introduction to Evaluation of Rockslope Deterioration

On the basis of field and laboratory work undertaken and presented in this thesis, a new classification is proposed, called Rockslope Deterioration Assessment (RDA). The primary aims of RDA are:

(i) to provide a relative measure of the *risk* (probability and severity) of rockslope deterioration;
(ii) to enable assessment of the nature of the deterioration *hazard* which might arise;
(iii) to provide guidance on appropriate slope treatment and maintenance measures;
(iv) to assist in identifying the relative influence of a range of intrinsic and external factors on rockslope deterioration potential.

These aims can be largely achieved by the application of ratings to a range of factors influencing rockslope deterioration and by evaluation of rock mass structure. Interpretation of deterioration morphology can also assist in the assessment in the case of existing rockslopes. The method includes guidance on the timing and frequency of processes of deterioration. There is no attempt in RDA to *accurately* predict or *quantify* the risk or nature of deterioration. Even with impracticably detailed field and laboratory testing, it is doubtful that the current state of knowledge of rockslope processes is sufficiently advanced that this could be achieved anyway. RDA is applicable to any long term excavated slopes in rock such as road cuttings, disused quarry faces, and disused slopes within active quarries. It is not applicable to active quarry faces which are continually being excavated. With caution, RDA can be applied to natural slopes, except where they are being continually re-excavated by undercutting.

A brief outline of current approaches to the evaluation of rockslope deterioration was given in section 1.4 of Chapter One and will not be repeated here. However, it is useful to set classifications in context with other forms of slope assessment generally, and also to look at the basic aims, advantages and disadvantages of classification schemes.

8.1.1 Classification as a method of rockslope evaluation

Three different approaches to the evaluation of rockslope deterioration can be identified: (i) *Analytical solutions* either address static forces using limit equilibrium methods and finite element analysis, or kinematic forces using stereographic projection. Analytical solutions are described and explained in numerous publications including Hoek (1973), Burman et al (1975), Attewell and Farmer (1976), Hoek and Bray (1981), Matheson (1983a, 1988, 1991), Hencher (1987), Nash (1987), Walton (1988), Giani (1992); Richards (1992). Most commonly, the potential for failure is determined in the context of a factor of safety (section 6.2.2.1). However, these methods are often not appropriate for evaluation of deterioration because the mechanisms involved are difficult to quantify, largely because they are not clearly understood or even identified in some cases. The quantity of field and laboratory testing which would be required to provide adequate data, therefore, would normally be unrealistic (Hack and Price 1993). (ii) *Observational methods* depend upon the measurement and monitoring of slope movements, and thus tend to be very site specific. Detailed process studies typified in the geomorphic literature contribute to an understanding of fundamental mechanisms, but this might not be widely or readily applicable. Observational methods often yield useful concepts based on the accumulation of experience and application of judgement. (iii) *Classifications* are an empirically based approach in which relationships between properties usually have some statistical or experiential basis, though can also be based entirely on observation. Classifications can be very widely applicable, are usually simple to apply and require relatively little data input. The drawback of classifications is that although developed primarily as a design tool, they are often misused, and applied in isolation from other methods. This can lead to over-simplification of investigations (Bieniawski 1989) and features or conditions of critical interest might not be identified.

8.1.2 Types of classification used in rockslope assessment

Three main types of classification are currently used in the assessment of rockslope stability:

Classifications of rock mass type and structure: Duncan and Goodman (1968) and Aydan et al (1992) have produced classifications which combine the influence of slope failure mode with rock mass structure. These were considered in section 7.3.5.1.

Rock mass classifications: Most rock mass classifications have been designed to evaluate support requirements for tunnelling and other underground excavations in rock (eg Barton et al 1974; Laubscher 1977; Bieniawski 1979; Kendorski et al 1983). Selby (1980) and Romana (1988, 1993) have developed rock mass classifications specifically designed to address slope instability, the former for natural slopes and the latter for excavated slopes. However, even where specifically designed or modified to address slope stability, rock mass classifications are not generally applicable to deterioration problems. This is illustrated very clearly by Ross and Reeves (1995) who applied several different rock mass classifications to igneous and metamorphic highway slopes in Scotland. They found that neither rock mass classifications nor conventional slope stability analyses matched the actual maintenance requirements of the slopes under consideration. They suggested the reason for this was because the maintenance requirements related largely to blast induced and weathering related fractures which were not taken account of in the various analytical and classification techniques applied.

Slope hazard or rockfall hazard systems: Several schemes have been developed which attempt to assess the hazard and risk of rockfalls and other slope failures in rock (Sinclair 1992; Mazzacola and Hudson 1996; Bunce et al 1997; Franklin and Senior 1997a, 1997b; Hack and Price 1997; McMillan and Matheson 1997, 1998). These are considered further in section 8.1.4.

8.1.3 General aims of classifications

There are several objectives which application of a classification should satisfy and these are illustrated with reference to RDA: RDA enables a form of qualitative sensitivity analysis in which properties or conditions most critical to deterioration potential and behaviour are identified. RDA also enables a rockslope to be sub-divided, as appropriate, into zones of similar deterioration behaviour and potential, and highlights zones requiring more detailed investigation or monitoring. The scheme assigns a risk class for a given range of characteristics and provides guidance on the practical meaning of those classes in terms of the nature of the hazard and its mitigation. Application of RDA enables a comparison of characteristics on different rockslopes and thus provides a common basis for communication (Bieniawski 1989). RDA also provides some quantitative data which can be used in slope design, deterioration mitigation and maintenance planning.

8.1.4 Concepts of *'risk'* and *'hazard'* in rockslope deterioration evaluation

Slope and rockfall hazard schemes have variously been used to determine the risk and/or hazard arising from slope failure. To avoid confusion, these terms are defined here in broad agreement with the IUGS Working Group on Landslides (1997), where **risk** is the probability and severity of a hazard occurring, and **hazard** is a condition with the potential to create an adverse impact. The IUGS recommend that hazard be described with reference to characteristics, magnitude and velocity, where appropriate.

Bunce et al (1997) developed a scheme by which the risk (ie the probability and severity) of rockfall on highway slopes could be *quantified* and compared with known risks from other hazards such as death in child birth, death due to drowning and plane crashes. They were able to do this because reliable and detailed frequency and magnitude data for past rockfalls were obtained from interpretation of road surface impact marks. This type of data is rarely available, however, and therefore estimates of risk are more usually qualitative and relative.

RDA is divided into three stages. Stage One involves the application of ratings to a range of rock mass and material properties and other external factors. Stage Two involves evaluation of the nature of deterioration and Stage Three provides guidance on its treatment.

In Stage One the ratings have been weighted such that higher values equate with an increased probability and severity of deterioration. Stage One, therefore, deals entirely with relative risk assessment. However, the emphasis in the context of deterioration is on severity rather than probability, since the probability of some deterioration occurring is a virtual certainty for all rockslopes.

Some rockfall hazard schemes have incorporated the assessment of consequences of slope failure into the risk element. The Oregon Rockfall Hazard Rating Scheme by Franklin and Senior (1997a, 1997b) and the rock slope hazard assessment method developed at the Transport Road Research Laboratory by McMillan and Matheson (1997, 1998) address the consequences of slope failure. They do this by incorporating information on traffic flow, sight lines, road width, and the likelihood, extent and duration of road blockage due to rockfall. The consequences of deterioration are not specifically addressed in RDA and it is recommended that they are dealt with on a slope by slope basis. Criteria for evaluating hazard consequences should be prescribed by the relevant highway authorities, landowners and organisations involved in the management and maintenance of slopes. As well as being related to the risk and hazard of deterioration, consequences are also a function of (i) the purpose of the slope (eg quarry, highway cutting, educational site), (ii) adjacent land use, (iii) vehicular and pedestrian access, (iv) the nature of potential casualties (eg human, animal, structures, vehicles), and (v) the proximity of the potential casualties to the hazard. Although consequences of deterioration are not specifically addressed in RDA, it is recognised that some parameters included in the ratings of Stage One, will nevertheless, influence the potential consequences (eg geometric factors affect rockfall trajectory) and can therefore be used in their evaluation.

Stage Two of RDA provides guidance on interpreting the likely nature of the deterioration hazard. Since deterioration modes have been classified according to constituent material size, velocity of movement and frequency of occurrence (section 7.3.4), the probability and severity of occurrence (ie risk) is also addressed. Stage Three of RDA combines the information derived in Stages One and Two to provide guidance on protective works, stabilisation measures and maintenance programmes to mitigate the deterioration hazard.

8.2 Structure of Rockslope Deterioration Assessment (RDA)

As explained in the previous section, RDA is divided into three stages, dealing with the risk, nature and mitigation of rockslope deterioration respectively. The structure and procedure for RDA is briefly described here and illustrated in Figure 8.1, before the components of each stage are explained more fully.

Stage One requires applying two sets of ratings to field data to produce an *adjusted RDA Class* (RDA_A hereafter). The first set of ratings apply to four key parameters, two of which, discontinuity spacing and aperture, relate to rock mass properties, and two of which, rock compressive strength and weathering grade, relate to material properties. The rock mass and material properties are weighted equally, giving a maximum *unadjusted RDA Rating* (RDA_U hereafter) of 100. The RDA_U Rating provides a relative measure of risk associated with the intrinsic properties of the excavated rock mass under consideration. In this respect it enables comparison of fundamental geological influences on rockslopes in different rock masses, without being complicated by the influences of a wide range of external factors.

The second set of ratings relates to the external factors and includes environmental conditions (altitude, exposure and climatic conditions; aspect; groundwater and surface runoff); stress conditions (static and dynamic stress); engineering factors (excavation method; stabilisation and protective measures); excavated slope characteristics (vegetation cover; slope geometry; rock mass structure); and other factors (time since excavation; direct disturbance). The ratings for these adjustments vary from -10 to +13, recognising that some influences will increase deterioration risk (positive values), while others will reduce deterioration risk (negative values).

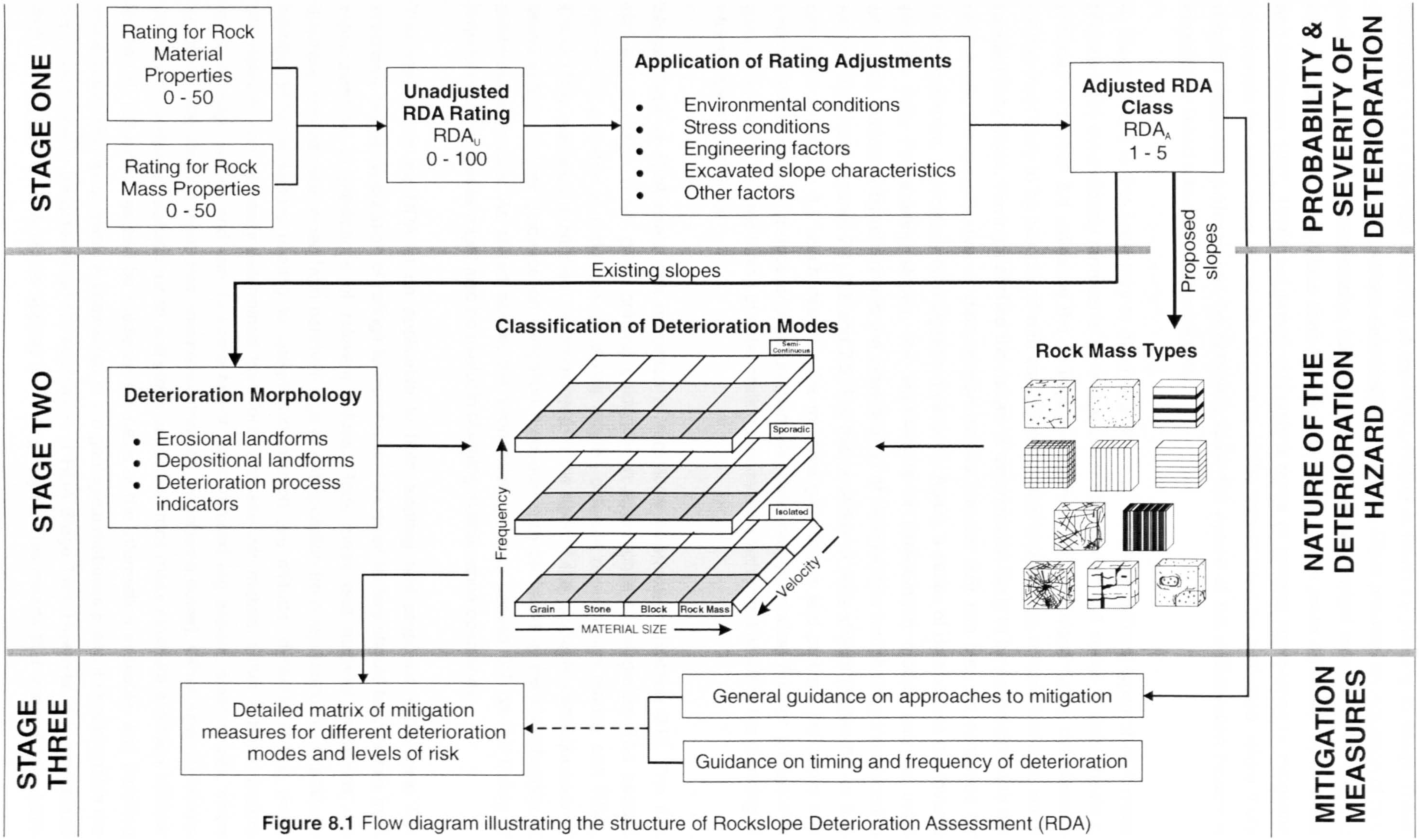


Figure 8.1 Flow diagram illustrating the structure of Rockslope Deterioration Assessment (RDA)

Guidance on the practical meaning of the RDA_A Rating is found by jumping to Stage Three, dealing with mitigation of rockslope deterioration. The RDA_A Class provides an indication of risk, particularly severity, of deterioration, taking account of all the principal external influences and controls upon it. The RDA_A Class does not prioritise action as with some schemes (eg McMillan and Matheson 1997, 1998), but rather, suggests a range of general approaches to mitigation appropriate for each class. However, the nature of the hazard must be evaluated before more detailed mitigation guidance can be obtained, since the nature of the deterioration hazard is inextricably linked with deterioration severity.

In Stage Two, it is first necessary to determine which of the seven primary types of rock mass (Figure 7.12) most closely represents the mass under consideration. A series of data sheets is provided in section 8.4 indicating the geological occurrence of rock mass types, deterioration modes most likely to be associated with each, geotechnical implications, notable features, and a typical RDA_A Class. Having identified the nature of deterioration likely to be associated with the rock mass, the classification of deterioration modes (section 8.5) can then be consulted for further guidance, particularly on mitigation measures. Again, a series of data sheets is provided (section 8.5). For existing slopes, the assessment of deterioration modes can be more accurately evaluated by referring to the classification of deterioration morphology introduced in section 7.3.3 and Figures 7.1, 7.2 and 7.3. For this, a series of data sheets (Plates 8.1 to 8.8) are given in section 8.6 which describe the morphological forms and process indicators and indicate their likely occurrence and association with deterioration modes. Further guidance is given in section 8.7 on the timing of deterioration processes which might assist in the planning of maintenance operations.

Stage Three of RDA deals with mitigation of rockslope deterioration (section 8.8). The first element of Stage Three gives general guidance on approaches to mitigation. The second element is a matrix of mitigation measures relating different deterioration modes with RDA_A Class. This guidance must then be considered in the light of the probable consequences of deterioration. Further guidance on mitigation measures can be found in the classification of deterioration modes. As stated above, the information given in section 8.7 on the timing of deterioration processes might also be useful in planning maintenance operations.

The intention is for RDA to be applicable to both existing and proposed rockslopes. For proposed slopes, application of ratings to intrinsic and external factors should be possible from measurements or predictions of relevant values. Rock mass and material properties, for example, can be determined from borehole logs or extrapolation from representative exposures nearby. External factors relating to geographic location (eg altitude, dynamic stress, direct disturbance) can be easily determined from local and site information. Other external factors relate to slope design and can, to some extent, be controlled (eg aspect, static stress, slope geometry, stabilisation measures, excavation method, vegetation cover). Other external factors such as groundwater, surface runoff and some aspects of rock mass structure are more difficult to predict, but estimates can be made on the basis of the information available, and modified immediately after excavation. A knowledge of the geological materials present should enable the rock mass type to be determined for application of RDA Stage Two. However, it is recognised that in many cases, RDA will be applied to existing slopes. This means that many of the above rockslope characteristics and influences can be more accurately verified. The influence of time since excavation can also be accounted for. Of greater interest though, is the fact that deterioration which has occurred since excavation will provide evidence to be incorporated into the assessment. Thus the classification of deterioration morphology is the one element of RDA specifically developed for use with existing rockslopes only.

ROCKSLOPE DETERIORATION ASSESSMENT: STAGE ONE

8.3 A Ratings Approach to the Evaluation of Rockslope Deterioration

Several basic principles of established rock mass classifications have been utilised in development of RDA. The unadjusted RDA_U Rating, for example, will lie between 0 and 100 and is comparable with the Rock Mass Rating (Bieniawski 1979) in this respect. This enables straightforward classification on a sliding scale in which relative risk is easy to envisage. In theory, an adjusted RDA_A could be produced which is in excess of 100, or less than 0, but in practice this is extremely unlikely to occur. A five-fold class system is also used in which class boundaries, illustrated in Figure 8.2, are evenly distributed between 0 and 100. Class 1, allocated for RDA_A of 0 to 20, represents a rockslope with a very low risk of deterioration, and class 5, for RDA_A of 80 to 100, represents a rockslope with a very high risk of deterioration.

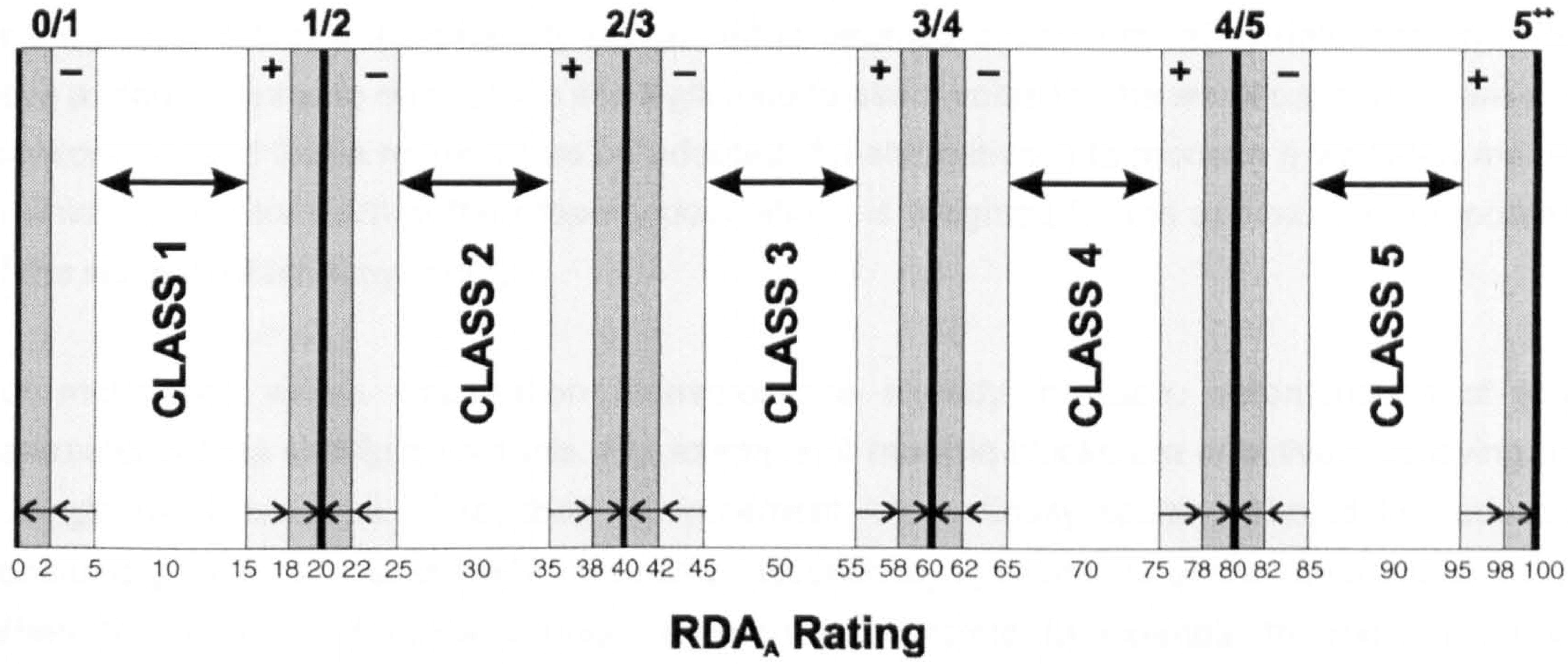


Figure 8.2 Illustration of RDA_A Class boundaries

Notes: If RDA_A lies within 2 points of a class boundary, use both classes to describe. If RDA_A lies within 3 to 5 points of a class boundary, use a '+' or '-' as appropriate. Where the RDA_A lies exactly on any boundary shown, move in the direction of the small arrows (eg 25 = 2-; 35 = 2+; 42 = 2/3; 58 = 3/4).

Each key parameter and adjustment factor is given a weighting which reflects its importance in deterioration. Over 200 slope units have been assessed using RDA (results are presented in Chapter Nine) and results suggest that the weightings used work well and are a good reflection of deterioration observed in the field.

8.3.1 Collection of data

A proforma for data collection and analysis is provided in Figure 8.3. Prior to application of RDA a preliminary appraisal should be undertaken to sub-divide the rockslope into zones of similar character. Further sub-divisions might become apparent during data collection. The possibility of identifying such zones in proposed slopes will depend upon the nature and quality of the information available. In a rockslope containing distinct zones each should then be assessed separately and a unique RDA Rating established for each. This will enable evaluation of deterioration risk for each zone, giving a more targeted approach to mitigation and maintenance. In some rockslopes, variable zones might not be sufficiently distinct to assess separately. In this case, several options are possible: (i) RDA can be applied to the groups of properties which are most common (ie those covering a larger proportion of the slope). (ii) However, if these variable zones or groups of properties are equally distributed, RDA could be based on those which appear to be contributing more to deterioration. (iii) If the slope is new, mean values can be adopted. (iv) Mean values are relatively meaningless, however, and so RDA could alternately be applied to, depending on circumstances, to the best or worst case. In any event, the relevant adjustments (section 8.3.3) for rock mass variability should be applied as necessary (eg K2.c).

It might prove difficult to assign a rating to a single property which is either highly variable, or where distinct populations exist. Again, the options are to take the worst case, the mean value, or the modal value. If the property concerned is deemed to be dominating deterioration or to have a high potential to do so, then it is legitimate to use a value for the worst case. Otherwise, it is recommended that a modal value be adopted. An alternative is to produce a weighted mean, in which ratings for each of the property populations is weighted for the approximate proportion of the slope in which they occur.

For rockslopes where stabilisation measures are already in place, determination of key parameter values should reflect this. For example, if multiple blocks are effectively behaving as a single block because of rockbolt reinforcement, discontinuity spacing should be reduced accordingly, or if fractures have been sealed, discontinuity aperture should be regarded as zero. Where the surface of the rock has been covered, by shotcrete, for example, the rock should be classified as unweathered (fresh) and intact (with no discontinuities). Where mitigation measures merely reduce the consequences of deterioration (eg netting, rocktrap ditch), values for key parameters should be determined in the normal way.

8.3.2 Selection and weighting of key parameters

Ratings for the four key parameters considered below are weighted in accordance with their potential influence on rockslope deterioration. Higher scores indicate a larger contribution to deterioration risk. This is converse to many rock mass classifications where a higher score indicates greater rock mass quality (eg Bieniawski 1979; Selby 1980).

FIELD DATA COLLECTION

Site Name and GR: Slope unit:

Material properties *(colour, grain size, texture, fabric, strength, weathering grade, lithology)*

Mass properties *(fracture spacing, fracture type and aperture, orientation, persistence)*

Environmental conditions
Altitude: Exposure level: Aspect:
Frost, moisture or sun pocket:
Groundwater and surface runoff:

Stress conditions
Excavation depth *(proposed slopes only)*: Surcharge:
Dynamic stresses:

Engineering factors
Excavation method *(proposed slopes only)*:
Existing stabilisation measures:

Excavated slope characteristics
Vegetation cover:

Slope geometry *(benching and uniformity, slope height)*:

Rock mass structure *(intersections, interlocking, variability, discontinuity favourability)*:

Other factors
Time since excavation:
Landuse *(direct disturbance)*:

EVALUATION OF RDA RATING

		Value	Rating
A:	Fracture spacing		
B:	Fracture aperture		
C:	Rock strength		
D:	Weathering grade		

Unadjusted RDA_U Rating = A + B + C + D = $\boxed{}$

General approach to mitigation:

	Code(s)	Descriptions(s)	Ratings(s)
Environmental conditions			
Stress conditions			
Engineering factors			
Excavated slope			
Other factors			
Total adjustment			

Adjusted RDA_A Rating = RDA_U + Total adjustment = $\boxed{}$

Deterioration morphology *(erosional and depositional landforms, process indicators)*

Rock mass type(s)
Likely deterioration mode(s)

Specific mitigation measures and maintenance requirements

Figure 8.3 Data collection proforma for Rockslope Deterioration Assessment (RDA)

8.3.2.1 Fracture spacing (block size)

The collection of data for discontinuity spacing differs from other classifications. Critically, any estimate of spacing should include ALL fractures, whether they form repeated fracture sets or not, and whether they are open or have a tight aperture, though they must be open to some extent (ie with a tensile strength less than that of intact rock). This means that when identifying fractures, those induced by excavation, weathering, rebound or even other anthropogenic causes should be included alongside bedding planes, joints, faults and other lithologically related fractures. Contrary to advice given in the literature (eg Selby 1980), incipient or 'hairline' cracks should not be ignored. These so-called 'superficial fissures' (Selby 1980) might not have a significant influence on rock mass strength, but have a very significant influence on deterioration potential. It is possible to obtain a reasonably accurate measure of discontinuity spacing using standard scanline survey techniques, especially if several are conducted at orthogonal angles. However, this is often not possible. An alternate method, though less accurate, is to estimate the mean block dimension on the basis of observation, supplemented with short scanlines. The maximum rating which can be given for fracture spacing is 35 and can be determined from Figure 8.4.

8.3.2.2 Fracture aperture

Ratings applied to fracture aperture are based on rather different criteria than those for other rock mass classification schemes. For instance, the RMR (Bieniawski 1979) incorporates aperture as one element of four defining discontinuity condition. The selection of these elements largely relates to their influence on shear strength of discontinuities, a critical factor in slope stability. In RDA, much greater emphasis is placed on the role of fracture aperture in (i) weathering (allowing root growth, water flow, block wedging, wall weathering and dissolution); (ii) providing potential chutes for material re-distribution; and (iii) contributing to the general 'looseness' of the rock mass. It is argued that once a fracture is open, its walls are immediately vulnerable to weathering, and therefore increasing the size of aperture has relatively little impact. It is for this reason that the largest increases in ratings occur at the narrower end of the scale, and big increases in apertures which are already large, attract little additional numerical weighting. Aperture should be measured in relation to separation of fracture walls, regardless of any infilling, unless the fracture is healed. In this case, it should be regarded as intact. The maximum rating which can be given for fracture aperture is 15 and can be determined from Figure 8.5. The maximum total rating for rock mass properties is 50.

8.3.2.3 Rock compressive strength

The intact strength of rock is regarded as being of lesser importance in rock mass classifications concerned with slope or tunnel stability than discontinuity spacing (Hack and Price 1993). This can be seen from the RMR system (Bieniawski (1979) in which a maximum of 70 points can be allocated for discontinuity spacing and condition, with only 15 being allocated to rock strength. In considering deterioration, however, rock strength takes on equal importance since weathering and erosion processes attack rock material as much as the discontinuities contained within it. Although rock compressive strength is a measure of resistance to crushing force, it also closely

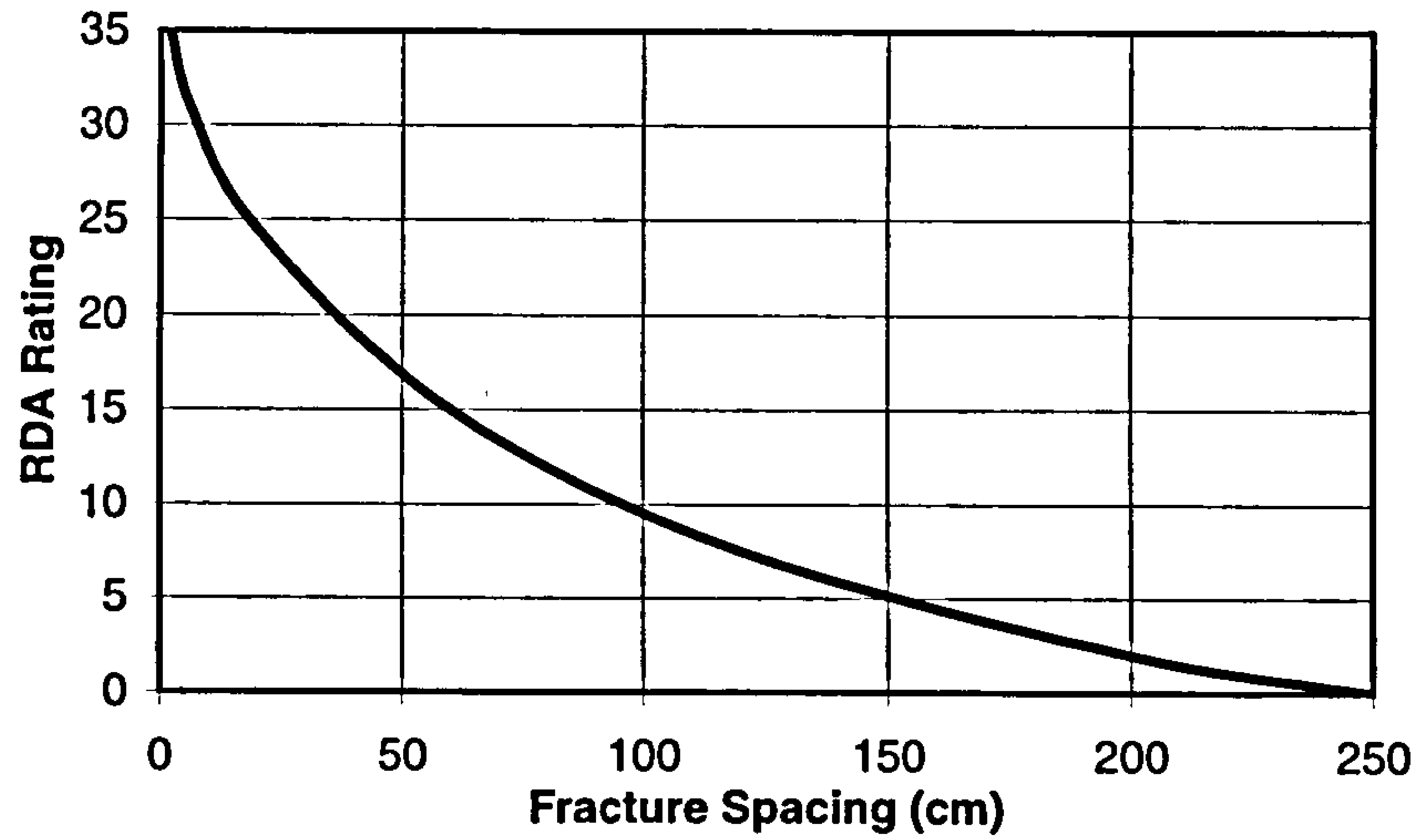


Figure 8.4 RDA rating curve for fracture spacing

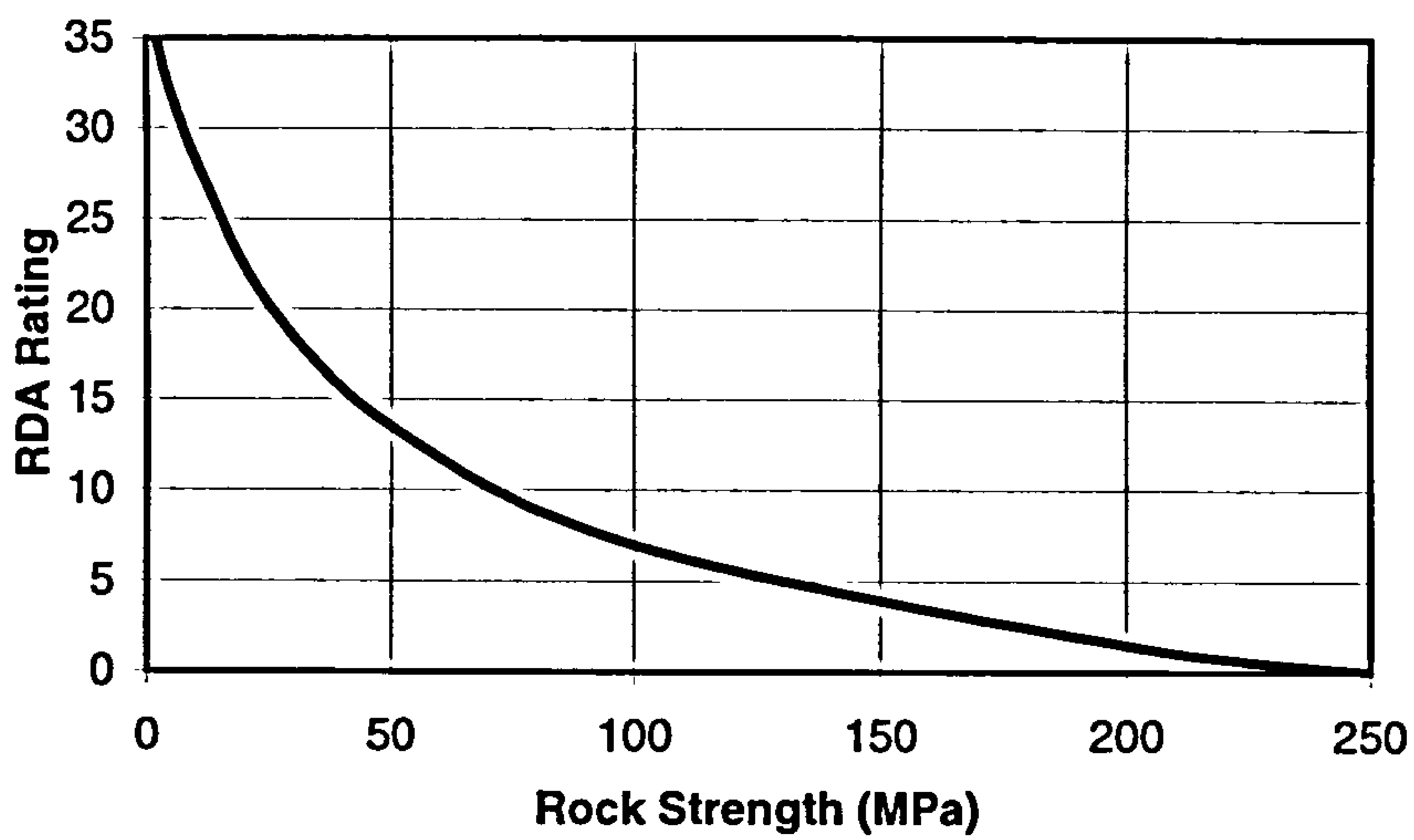


Figure 8.6 RDA rating curve for rock strength

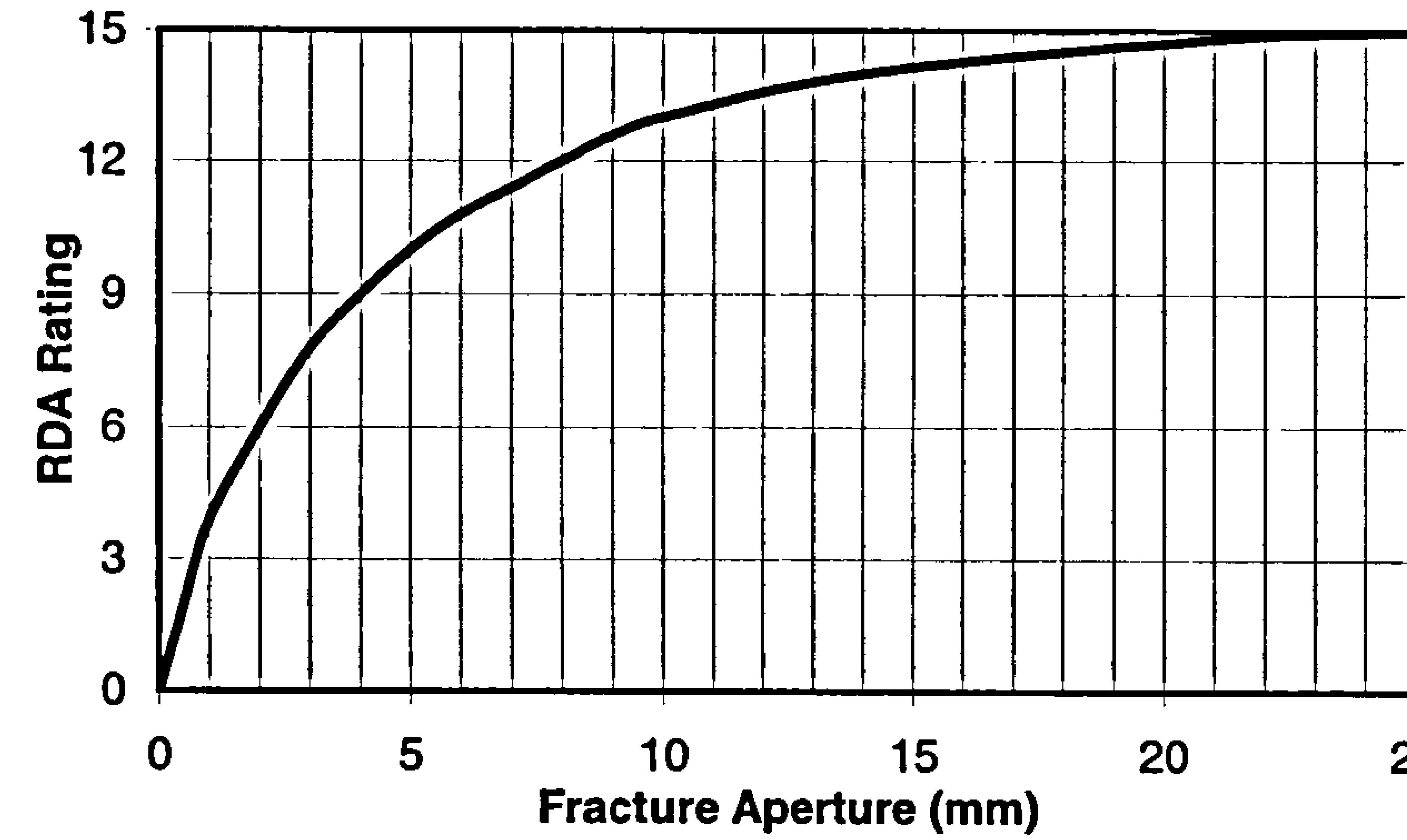


Figure 8.5 RDA rating curve for fracture aperture

Grade	Material description	Rating
Fresh	Unchanged from original state.	0
Slightly Weathered	Slightly discoloured, slight weakening.	5
Moderately Weathered	Weakened in association with penetrative discoloration.	10
Highly Weathered	Large pieces (eg NX drill core) cannot be broken by hand; does not readily slake when immersed.	14
Completely Weathered	Considerably weakened; slakes readily; original texture retained.	15
Residual Soil	No original fabric or texture remains (soil).	n/a

Table 8.1 RDA ratings for material weathering grade

relates to other rock properties such as texture, density and porosity, and is a reasonable surrogate for rock durability (refer to Chapters Four and Five, and section 6.2.1.2). Rock compressive strength should be estimated from the field-based guide presented in BS 5930 (1999). It can be helpful to obtain Schmidt hammer rebound to refine the estimate, but this value should not be used as the sole measure of strength. Point load testing can also be undertaken to obtain a more accurate field estimate of compressive strength if portable equipment is available and field measurements can be improved if more accurate laboratory test data become available subsequently. The maximum rating which can be given for rock strength is 35 and can be determined from Figure 8.6.

8.3.2.4 Rock material weathering grade

Ratings for weathering grade should be applied on the basis of a qualitative assessment of the weathering condition of the rock material as a whole. Although material weathering might relate to discontinuity walls as much as to the intact rock, it should be based on the latter. The classification system used is largely based on the scheme developed by Moye (1955) to describe the weathering state of granites. It should not be used in the sense of Martin and Hencher (1986) to define the proportion of weathered to unweathered rock, or of Fookes et al (1971) to consider the combined effects of weathering on mass and material properties. The maximum rating which can be given for rock weathering grade is 15 and can be determined from Table 8.1. The maximum total rating for rock material properties is 50.

8.3.3 Rating adjustments

Rating adjustments are made on the basis of the potential influence of various conditions on rockslope deterioration. The ratings carry the presumption of a 'standard' rockslope, where positive and negative adjustments are made for non-standard conditions. A 'standard' slope is considered to be an existing rockslope situated in a protected, low altitude location in a marine temperate climate (eg United Kingdom, or comparable). It is not subject to any significant dynamic or unusual static stresses and is less than 15m in height. The slope is dry, has no vegetation cover, stabilisation or treatment works, and is not subject to any direct disturbance. The slope is new, and its structure neatly fits into one of the seven rock mass types.

In most cases, many of the adjustments will not apply. A typical, total adjustment will lie in the range -5 to +15 (Figure 9.2), but could, in rare circumstances, be as much as -25 or +25 (ie more than one RDA Class). The RDA Rating adjustments serve two primary purposes, (i) to adjust the RDA_U Class such that it reflects the most likely risk of deterioration, and (ii) to draw attention to particular external factors which influence deterioration behaviour. Considerable judgement is needed in applying adjustments.

There are five groups of adjustments, pertaining to environmental conditions, stress conditions, engineering factors, excavated slope characteristics and other factors. Each of these is sub-divided, each sub-section being denoted with an upper case letter (eg under section 8.3.3.1 environmental conditions, sub-section A relates to altitude, exposure and climatic conditions). There are 12 sub-sections in total. Most of the ratings are designed such they can be applied to either proposed or existing rockslopes. Some adjustments, such as those for deep excavations (D1) and excavation method (F) should be ignored for existing rockslopes, while others, such as stabilisation and protective measures (G1) and slope geometry (J2.a and J2.b) ONLY apply to existing slopes. For each sub-section, no more than one adjustment should be used for each group of similarly numbered items. For example, in sub-section J: Slope Geometry, adjustments 1.a and 2.b could both be applied, but 1.a and 1.c could not. Adjustments are all positive (in other words deemed to contribute to deterioration) unless otherwise stated. The rating scheme for adjustment factors is presented with explanatory notes below.

8.3.3.1 Environmental conditions

A	**Altitude, Exposure and Climatic Conditions**	
1.a	High altitude (>300m) localities.	Up to 4
1.b	High altitude localities or coastal locations which are also slightly, moderately or very exposed (affected by *driving* wind and rain).	2 to 7
1.c	Moderately or very exposed, moderate altitude (150 to 300m) localities.	1 to 3
1.d	Moderately or very exposed, low altitude (<150m) localities.	1 to 3
2.a	Frost or moisture pockets and sites of cold air drainage. *Apply only to slopes which are very sheltered and enclosed. Use a greater adjustment for high slopes (eg >12m).*	1 to 4
2.b	Sun traps. *Apply only to low (eg <8m high), south or south west facing slopes which are sheltered or slightly exposed, and which are never shaded (eg by trees or structures). Use a higher adjustment for shallow gradient slopes or where the rock is rich in clays.*	Up to 3

Adjustment A1: Exposure levels are defined in section 7.2.4.3. Altitude can be obtained from topographic maps. For A1.a it is recommended that an adjustment of 3 be applied to slopes approaching 400m and the maximum adjustment be applied for slopes at significantly higher altitudes. For A1.b the adjustment should be based on the combined effects of altitude and exposure conditions.

Adjustment A2.a: Frost or moisture pockets are characterised by being permanently or semi-permanently cast in shade such that the rock rarely, if ever, dries out. This condition is likeliest for north facing slopes, in deep hollows, or in shallow hollows with a dense vegetation cover. Cold air drainage occurs where local conditions funnel and pond cold air draining from exposed slopes above, and is commonly found where tunnels, caves or mine entrances are present at a low level in or near the slope.

Adjustment A2.b: A sun trap is a slope situated in a sheltered environment which rarely receives the full force of wind and rain. It is also south facing and largely free from vegetation so that it receives maximum solar radiation. Shallower gradient slopes receive greater solar radiation intensity because of the angle of incidence of the sun. The primary effect of a sun trap is to produce rapid drying out of slope materials which can lead to drought. This can kill off vegetation periodically, which then collapses, taking slope materials with it. In clay-rich materials, it can also lead to desiccation cracking. The most serious deterioration effects occur if meteorological conditions are such that rapid and repeated wetting and drying occurs, especially in clay-rich rock.

B	**Aspect**	
	An aspect adjustment should not be applied for sheltered slopes, or for slopes where microclimatic behaviour is dominated by the position of the slope in a frost pocket or sun trap. Where an intermediate bearing applies, use an intermediate adjustment.	
1.a	Northerly aspect *(apply the higher adjustment where there is widespread evidence of rock moisture retention – eg moss, algae, surface staining).*	2 or 4
1.b	Easterly aspect *(ditto above).*	1 or 2
1.c	Southerly aspect *(apply a higher adjustment for slopes with vegetation cover).*	1 to 3
1.d	Westerly aspect.	0

Adjustment B1: Since aspect is indirectly incorporated into adjustments A2.a and A2.b, no duplication of ratings should occur. Where several slope aspects are represented, determine RDA Class separately for each, or use an intermediate rating. An adjustment of at least 1 should generally be applied for south westerly aspects.

C	**Groundwater and Surface Runoff**	
	Apply a lower adjustment where flow is very localised and a higher value if widespread. Apply the adjustment for the worst case. If contrasting volumes of flow are present in different areas of the slope, either (i) use an intermediate adjustment, or (ii) conduct a separate RDA evaluation. Note that flow need not be due to natural hydrological processes, but can be induced by failed slope drains or moisture retention due to artificial surface cover (eg masonry). In each case, conditions might be real or inferred from evidence. Refer to note in H.	
1.a	None, or very minor, localised surface dampness.	0 to 1
1.b	Slow dripping or moderate areas of damp or wet rock.	1 to 2
1.c	Steady dripping, light continuous flow or large areas of wet rock surface.	1 to 4
1.d	Moderate, continuous flow.	3 to 7
1.e	Excessive, continuous, strong flow.	4 to 9

Adjustment C1: It is recognised that direct evidence of groundwater seepage and surface runoff might not be available during dry periods, but *circumstantial evidence* of these can be gathered in the form of process indicators (section 7.3.3.3 and Plates 8.5, 8.6 and 8.7).

8.3.3.2 Stress conditions

D	**Static Stress**	
1	Deep excavations (eg where overburden depth removed >20m) in strong, massive rock. *This adjustment should be ignored for EXISTING rockslopes. Apply the maximum adjustment for very deep excavations (>50m).*	Up to 3
2	Surcharge at the slope crest (eg structures, trees). *Refer to note in H3c.*	Up to 2

Adjustment D1: The potential for stress release is much less in existing rockslopes since elastic rebound will have occurred at the time of excavation. Non-recoverable rebound will occur time dependently in response to weathering. This adjustment, therefore, only applies to proposed rockslopes.

Adjustment D2: In the context of excavated rockslopes, surcharge due to loading at the crest is uncommon, and is most likely to relate to normal stresses produced from mature trees.

E	**Dynamic Stress**	
1.a	Dynamic loading due to quarry blasting in close proximity (up to 100m) to the slope. *Apply a higher adjustment for long term, high frequency blasting.*	1 to 3
1.b	Ground vibration due to traffic movement on high speed roads in very close proximity (up to 10m) to the slope. *Apply the maximum adjustment for roads with a high proportion (>10%) of heavy vehicles AND poor road surface condition (eg rough, irregular, potholes and infilled trenches). Apply a lower adjustment where only one of these apply and no adjustment where neither apply.*	2 to 4

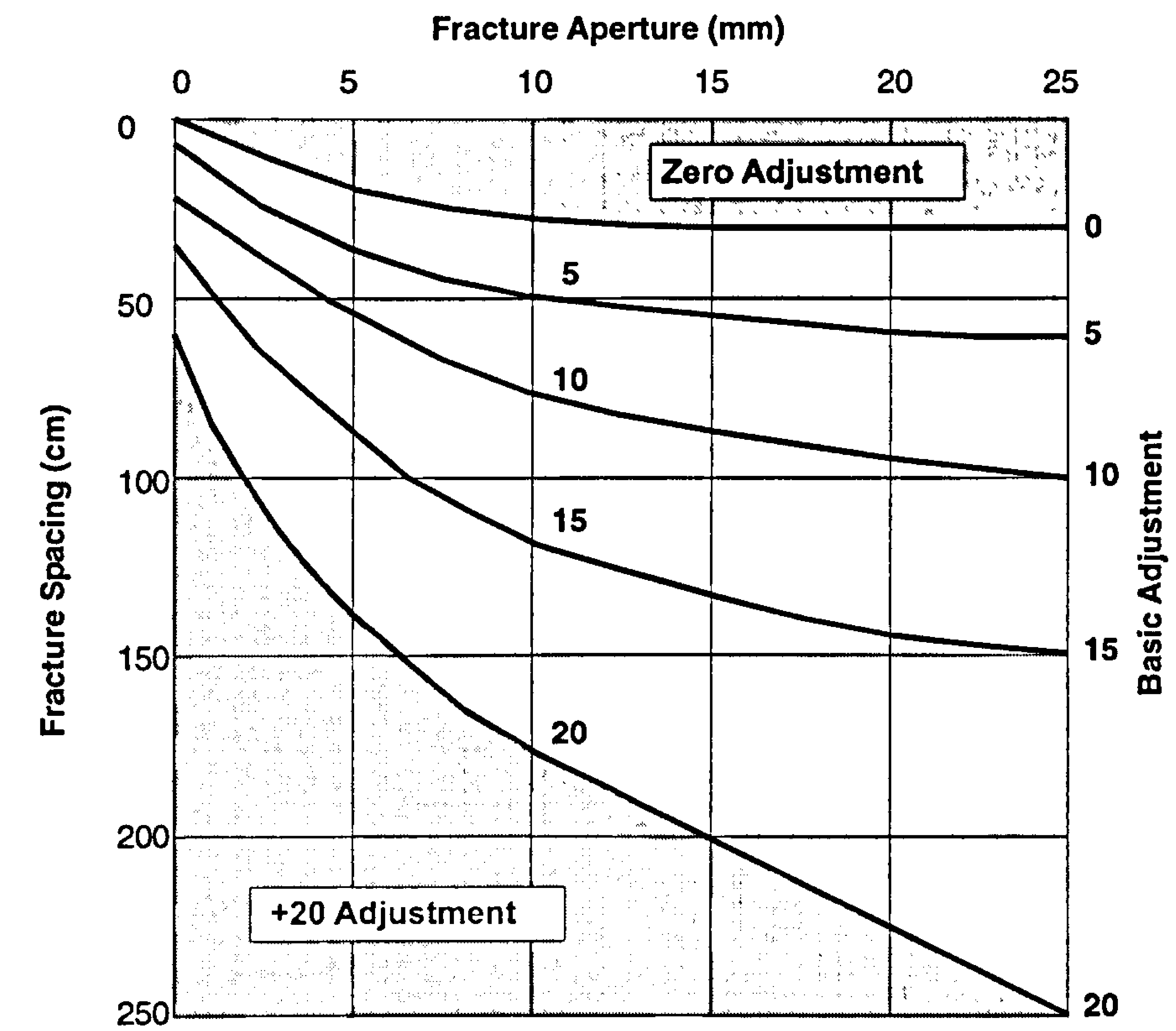
Adjustment E1: These adjustments are closely related to land use. E1.a is most likely for disused slopes within active quarries, although road cuttings or disused quarries in close proximity to active workings might also be affected. E1.b clearly relates to high speed roads such as motorways, dual carriageways, and some single carriageways in national (ie maximum) speed limit zones.

8.3.3.3 Engineering factors

F	**Excavation Method**
	Excavation method adjustments should be ignored for EXISTING rockslopes. For PROPOSED rockslopes, obtain an adjustment from the "RDA Rating adjustment for excavation method" (Figure 8.7).

Adjustment F: For existing rockslopes, any deleterious effect of excavation method will already have contributed to fracture spacing and aperture and there is no need to apply any adjustment. Adjustments should only be made for proposed slopes. The adjustments given in Figure 8.7 are based on two fundamental assumptions: (i) that some excavation methods have a much greater deleterious effect on rock mass quality than others, and (ii) that rock mass properties determine the effect of each excavation method. For example, the effects of bulk blasting (eg increases in fracture intensity and aperture) will be much more evident in a rock mass with few, tightly closed fractures, than in a highly jointed rock mass, where much of the blast energy will be dissipated via the existing fracture network.

Worked example: For a rockslope with a fracture spacing of 60cm and an aperture of 2mm, the basic adjustment from the chart would be +13. If the rock mass were to be excavated by bulk blasting, the *actual* adjustment to be added to RDA_U would be 13 x 0.9 = 11.7. If pre-splitting were used, the *actual* adjustment would be 13 x 0.2 = 2.6, and if hand excavation were used, 2.6 points would have to be subtracted from RDA_U. Some flexibility is given in the excavation method factors to allow for varying quality of blast design (eg hole spacing and charge weight), machinery and tools used.



EXCAVATION METHOD ADJUSTMENT FACTORS

1. For BULK (quarry) BLASTING add x0.8 to x1.0 of basic adjustment
2. For SMOOTH BLASTING add x0.3 to x0.6 of basic adjustment
3. For PRE-SPLIT BLASTING add x0.0 to x0.2 of basic adjustment
4. For MECHANICAL EXCAVATION add x0.0 to x0.1 of basic adjustment
5. For HAND EXCAVATION _subtract_ x0.2 to 0.0 of basic adjustment

Figure 8.7 RDA$_U$ Rating adjustment for excavation method (proposed slopes only)

G	Stabilisation and Protective Measures	
1	Deterioration associated with EXISTING stabilisation measures (eg material weathering around rockbolt heads; spalling associated with drainholes, rockbolts and dowels). *Apply a low adjustment unless a widespread effect.*	Up to 3

Adjustment G1: Occasionally, stabilisation measures exacerbate deterioration locally and the purpose of this adjustment is to allow for this to be accounted for in the RDA Rating.

8.3.3.4 Excavated slope characteristics

H Vegetation Cover

Care should be taken in H1a and H3a not to duplicate adjustments already made in C for surface moisture retention (eg by moss and algae) and its potential weathering effects.

Highly weathered or soil-like slopes (excluding highly fractured rockslopes)		
1.a	Cover of grass or other fine-rooted, low-growing, herbaceous plants. *Apply a maximum adjustment for widespread, dense, well established cover, an intermediate adjustment for moderate coverage of medium density vegetation, and a minimum adjustment for sporadic, thin or newly established cover.*	-10 to -2
1.b	Small woody shrubs and trees (up to 3m height). *Apply a high negative adjustment for widespread, dense cover and a low adjustment for isolated occurrences with only localised effect.*	-6 to 0
1.c	Widespread, dense cover of large woody shrubs and trees (exceeding 3m height). *Apply a high negative adjustment for vegetation up to 5m height and a low adjustment where this height is exceeded.*	-4 to -2
Where more than one adjustment applies in 1, use the largest relevant adjustment ONLY.		
2	Isolated growth of large woody shrubs and trees (exceeding 3m height). *Apply a low adjustment for vegetation up to 5m height and a higher adjustment where this height is exceeded.*	0 to 2
Slopes cut in rock (including highly fractured rockslopes)		
3.a	Cover of grass or other fine-rooted, low-growing herbaceous plants. *Use a low adjustment where growth is sporadic and a high adjustment for widespread cover or where substantial soil has accumulated.*	0 to 3
3.b	Small woody shrubs and trees (up to 3m height). *Apply a low adjustment for isolated occurrences with only localised effect and a high adjustment for more widespread cover or where substantial soil has accumulated.*	1 to 5
3.c	Large woody shrubs and trees (exceeding 3m height). *Apply a low adjustment for isolated occurrences with only localised effect and a high adjustment where there is more widespread cover, or substantial soil accumulation, or large stem diameter (greater than 20cm), or isolated occurrence with widespread effect. Where an adjustment was made for surcharge due to trees at the slope crest, the total adjustment for D2 and H3c should not exceed 8.*	2 to 7

Where more than one adjustment applies in 3, use the largest relevant adjustment ONLY.

Adjustment H: The underlying principles for vegetation cover adjustment are that (i) root growth in very weak and soil-like materials is likely to have a beneficial, reinforcing effect. This will be enhanced with widespread vegetation cover, particularly for grasses, herbaceous plants and low-lying shrubs. Growth of very large woody plants can lead to disruption due to windloading. (ii) Vegetation growth in stronger materials is likely to have a deleterious effect. This will be increased with widespread vegetation cover, particularly for taller plants and the more penetrative root systems of shrubs and trees.

For each of the conditions presented here a wide range of ratings is offered. This reflects the variety of vegetation effects depending on plant species; rooting depth and spread; age; nutrient availability; and moisture and temperature regime.

J Slope Geometry

1.a	Slopes (or individual risers in benched excavations) with a height greater than 15m. *Apply the maximum adjustment for slopes exceeding 30m in height. The total adjustment from items D1 and J1 should not exceed 5.*	2 to 5
1.b	Slopes wholly or partly comprised of benches less than 1.5m in height. *Only apply where total slope height is <15m and overall slope gradient is <60°.*	-2 to –5
1.c	Slopes with a total height less than 4m.	-2 to –3
2.a	Uniform, planar surface with little irregularity or large scale roughness in highly weathered or soil-like materials. *Apply only to EXISTING slopes.*	Up to 3
2.b	Uniform, planar surface with little irregularity or large scale roughness on slopes cut in rock. *Apply only to EXISTING slopes.*	-3 to 0

<u>Adjustment J1:</u> Where a rockslope contains several benches, each can generally be treated as a separate unit. However, some adjustments need to be made with respect to the slope as a whole (eg static stress and surcharge).

<u>Adjustment J2:</u> A uniform, planar slope profile will increase the velocity of surface runoff, and in very weak materials this can lead to wash erosion. In stronger rocks, undermining, overhang collapses, toppling and the like are less common on planar slopes.

K Rock Mass Structure

For rock masses with extremely poor material properties (eg highly weathered and weak) but excellent mass properties (eg essentially structureless) and vice versa (eg an intensely fractured and loose rock mass in strong, fresh rock), <u>and</u> where the RDA_U Rating for the two unfavourable parameters is >35, the total RDA_U Rating must be adjusted as in item 1. The adjustment to be applied can be determined from the chart given in Figure 8.8 (it can also be calculated using the percentage figures given in 1.a, 1.b and 2.c below).

1.a	Where the RDA_U Rating for the two favourable parameters is <10% of the rating for the two unfavourable parameters.	13
1.b	Where the RDA_U Rating for the two favourable parameters is <20% of the rating for the two unfavourable parameters.	9
1.c	Where the RDA_U Rating for the two favourable parameters is <30% of the rating for the two unfavourable parameters.	5
2.a	A single, dominant set of regular fractures (eg bedding planes or joints) such that there are very few discontinuity intersections and a regular structure.	-1 to -7
2.b	Rock mass dominated by angular, blocky shapes with at least one acute angle (eg highly interlocking).	-3 to 0
3	Highly variable or composite rock mass containing discrete zones of contrasting rock mass and/or material properties (eg highly fractured areas, shear zones) which cannot be evaluated separately. *Use a **positive** adjustment where ratings for mass and material properties have been based on the <u>most favourable zones</u>, and a **negative** adjustment where they have been based on the <u>least favourable zones</u> (eg intensely fractured zones).*	0 to 7 OR -7 to 0
4	Favourability of dip angle and direction for recurring fracture sets (favourable = 0; unfavourable = 1-3; very unfavourable = 3-6). *For small blocks (eg <300mm) use lower end of range given, and for large block sizes (eg >600mm) use higher adjustment. Favourability refers to deterioration failures NOT deep-seated slope instability.*	Up to 6

<u>Adjustment K1:</u> It is possible to envisage a situation in which a rock is so weak and weathered, that in terms of the potential influence on deterioration, the presence or absence of fractures and

their apertures becomes largely irrelevant. The reverse situation can also occur, where a rock mass is so intensely fractured, with wide apertures, that the fact that the material is strong and fresh, is irrelevant. In either case, it is possible for the RDA_U Rating to be around 50, giving an RDA_U Class of 3. This would not be a good reflection of the high risk of deterioration for that rockslope and so the adjustments given in K1.a to K1.c should be used. A chart (Figure 8.8) is used to determine the adjustment that should be applied.

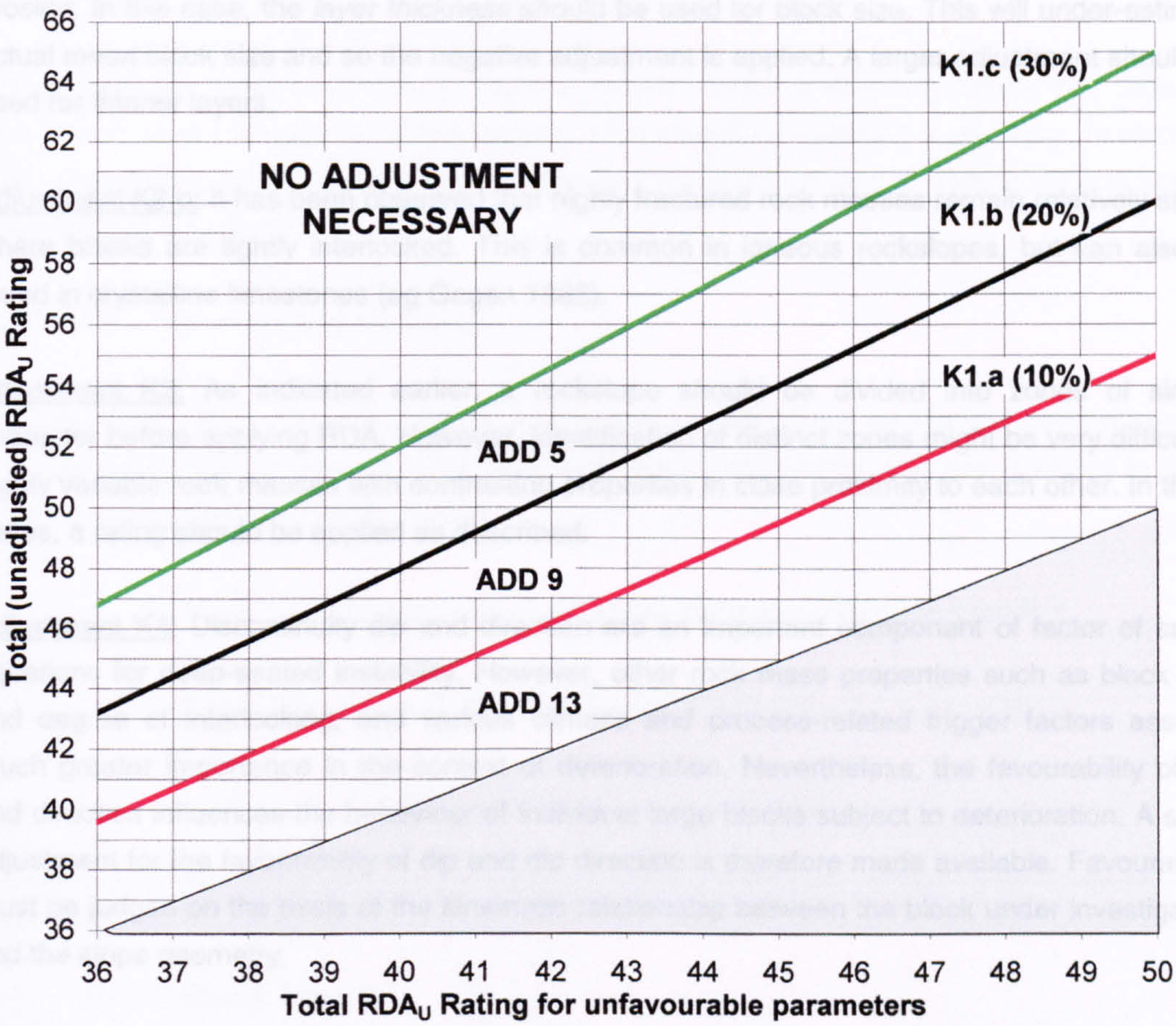


Figure 8.8 Chart K1: For use with RDA Rating adjustment K1a, b and c

Worked example: A rockslope has an RDA_U Rating of 55. This is made up of a mass rating (fracture spacing and aperture) of 47 and a material rating (rock strength and weathering grade) of 8. The fact that the rating for the two unfavourable parameters (ie the mass properties) exceeds 35 means that adjustment K1 is applicable. The chart given in Figure 8.8 can then be used to determine how much adjustment to make to RDA_U. Using the chart in Figure 8.8, the total unadjusted RDA_U Rating of 55 is selected on the y axis. This line is followed to the right until it intersects the x axis value relating to the total RDA_U Rating for the *unfavourable parameters*, which is 47 in this example. The intersection lies in the zone in which an adjustment of +9 should be made to the RDA_U Rating. This zone represents situations where the rating for the two *favourable parameters* (8 in this case) is between 10 and 20% of the rating for the two *unfavourable parameters* (47 in this case) and is described in section K1.b of adjustment K1.

K1.a deals with situations where the respective percentage is <10%, and K1.c deals with situations where the respective percentage is 20 to 30%.

Adjustment K2.a: In a layered rock mass in which the discontinuity causing the layering is extremely persistent and there are very few cross-cutting fractures, it can be difficult to determine block size. For thick layers (eg >1m), it is legitimate to calculate the *mean* block length. Since this is effectively infinite the maximum block size (2.5m) can be used. However, this is less acceptable for thinner layers since they are more vulnerable to weathering and erosion. In this case, the *layer thickness* should be used for block size. This will under-estimate actual mean block size and so the negative adjustment is applied. A larger adjustment should be used for thinner layers.

Adjustment K2.b: It has been observed that highly fractured rock masses remain relatively stable where blocks are tightly interlocked. This is common in igneous rockslopes, but can also be found in crystalline limestones (eg Gagen 1988).

Adjustment K3: As indicated earlier, a rockslope should be divided into zones of similar character before applying RDA. However, identification of distinct zones might be very difficult in highly variable rock masses with contrasting properties in close proximity to each other. In these cases, a rating should be applied as described.

Adjustment K4: Discontinuity dip and direction are an important component of factor of safety equations for deep-seated instability. However, other rock mass properties such as block size and degree of interlocking, and various climatic and process-related trigger factors assume much greater importance in the context of deterioration. Nevertheless, the favourability of dip and direction influences the behaviour of individual large blocks subject to deterioration. A small adjustment for the favourability of dip and dip direction is therefore made available. Favourability must be judged on the basis of the kinematic relationship between the block under investigation and the slope geometry.

8.3.3.5 Other factors

L	**Time Since Excavation**	
1.a	Natural slopes and slopes excavated more than 80 years ago. *Only adjust for natural slopes where there is no active erosion (eg by basal undercutting).*	-10 to -8
1.b	Slopes excavated between 50 and 80 years ago.	-8 to -5
1.c	Slopes 30-50 years.	-4 to -1
1.d	Pre-split blasted or slopes excavated mechanically 5 to 30 years ago.	-5 to -2
1.e	Pre-split blasted or slopes excavated mechanically in the last 5 years.	-2 to 0

Adjustment L1: The premise of this adjustment is that the severity of deterioration decreases with time since excavation, as slopes achieve equilibrium with their geological environment. Increased stability can also be achieved at an earlier stage if pre-split blasting or mechanical excavation methods are used.

M	Direct Disturbance	
1	Anthropogenic disturbance (eg walking, climbing, grazing, fossil collecting).	1 to 3
2	Basal undercutting (eg marine action; river erosion; weathering, collapse or erosion of underlying rock). *Use a higher adjustment if undercutting is rapid, or where there is evidence of collapse of overlying rock as a direct consequence.*	1 to 6

Adjustment M1: In most cases, human or animal disturbance of a rockslope is only likely to have a very localised influence on deterioration, but the range of ratings provided allows for unusual situations to be considered.

Adjustment M2: Basal undercutting, while a common process affecting natural sea cliffs and river banks, is rare for excavated rockslopes. One situation in which it does occur, however, is where substantial undermining of more competent rocks occurs due to weathering and erosion of underlying weak material.

8.3.4 Determination of RDA Class

The unadjusted RDA_U Class is determined from the sum of ratings for the four key parameters. The total amount of adjustment is the sum of all adjustment ratings, and this is added to RDA_U to determine the adjusted RDA_A Rating:

$$RDA_U = \Sigma \{ \text{Block size} + \text{aperture} + \text{rock strength} + \text{weathering grade} \}$$

$$\text{Total adjustment} = \Sigma \{A + B + C + D + F + G + H + J + K + L + M\}$$

$$RDA_A = RDA_U + \text{Total adjustment}$$

ROCKSLOPE DETERIORATION ASSESSMENT: STAGE TWO

8.4 Rock Mass Types

Seven types of rock mass were described in Chapter Seven (section 7.3.5), largely defined on the basis of the spatial distribution of open fractures and the rock mass structure. However, certain types of rock mass have a clear lithological association, and so in reality, both mass and material properties are represented in this classification. This can be seen in the detailed rock mass data sheets given in Figure 8.9 to 8.16. These give a brief description of the essential characteristics of the rock mass, its typical geological occurrence, deterioration modes most commonly associated with it and special features with implications for deterioration and treatment. Associated deterioration modes should be read in conjunction with Figure 8.17. Modes which are underlined are extremely or very likely to occur, those without underlining are likely to occur, and those described as 'minor' might occur. A typical RDA_A Class is also given for general guidance only. It is useful to compare measured RDA_A values against this typical value since a large deviation can indicate a particular slope condition requiring special attention.

WEAK MASSIVE

Description: Weak (eg <20MPa) rock masses with no dominant structure (ie essentially homogeneous), or with very wide fracture spacing. Might have occasional fractures or many closed discontinuities.

Occurrence: Occurs only in sedimentary rocks, usually those which are granular, friable, perhaps with poor cementing. Examples include weakly bonded sandstones (eg bonded with calcite, clay, gypsum); highly weathered gritstone and sandstone; mudstone; marl; weak chalk and weak oolitic limestone.

Special characteristics: Most deterioration relates to material breakdown and erosion. Slopes are prone to erosion by surface runoff and to penetrative material weathering, especially in damp environments. Might be vulnerable to wind erosion. Honeycomb weathering common in sandstones. Might behave like a soil if very weak. Reduction in groundwater and surface water flow critical for erosion control and minimisation of material weathering. Vegetation cover useful for ground reinforcement.

Associated deterioration modes: Grain ravelling, grainfall, wash erosion, contour scaling, (minor: *stonefall*).
Typical RDA_A Class: (Sed) **2/3+**. Rare in igneous and metamorphic rock.

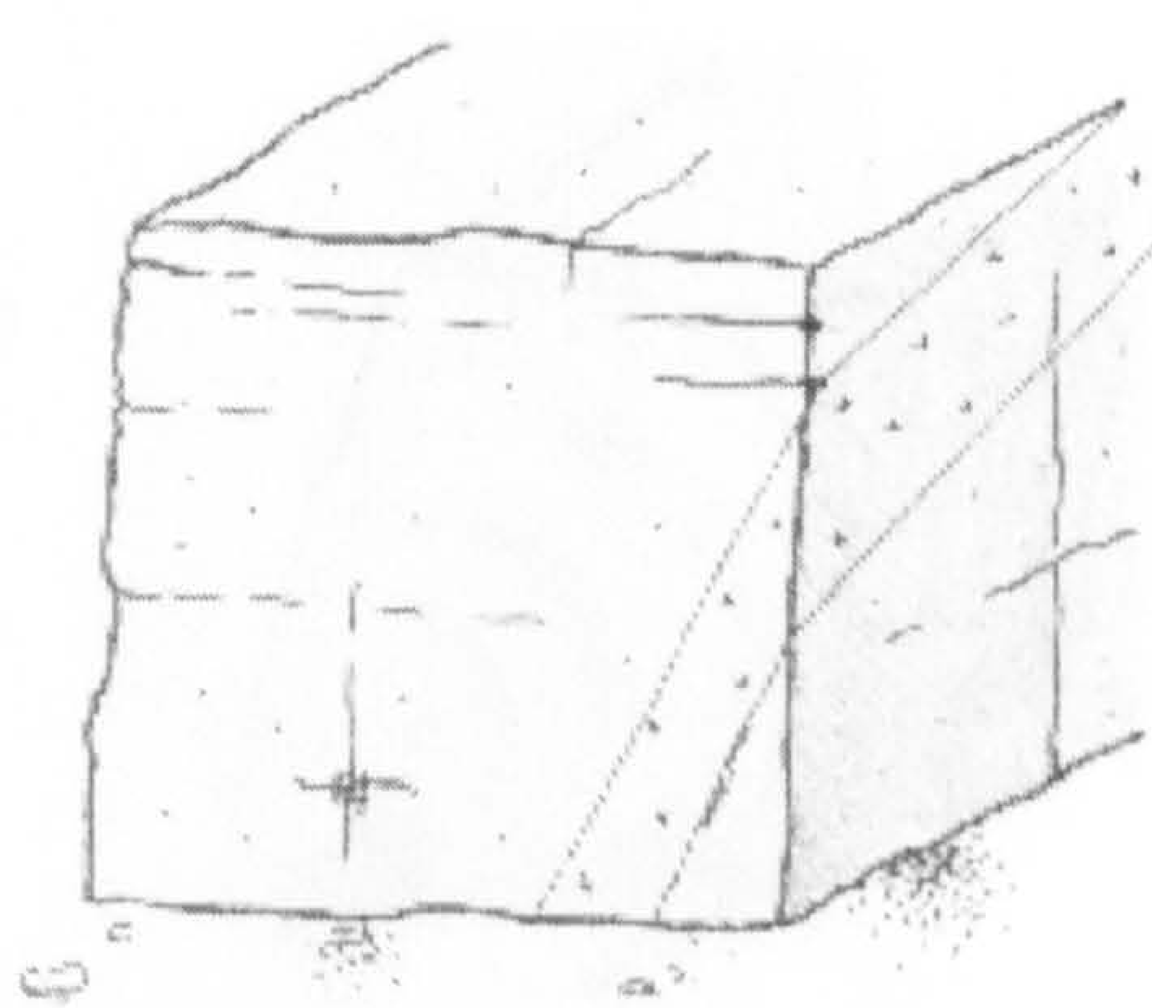
Figure 8.9 Classification of rock mass types: ***Weak massive***

STRONG MASSIVE

Description: Strong rock masses with no dominant structure (ie essentially homogeneous), or a very wide fracture spacing of around 1 to 3m (Deere 1968). Might have occasional fractures or many closed discontinuities such as fabric, laminations and bedding.

Occurrence: Occurs in a wide range of rock types, largely those unaffected by stress release, weathering and excavation induced fracturing. Examples include very thickly bedded gritstone and crystalline limestone, tough breccia and slightly metamorphosed sediments. Also structureless granite, basalt and tuff, and intact gneiss.

Special characteristics: Commonly resistant to deterioration except for localised wash erosion, scaling and fragmentation from local root wedging or solution. Might be susceptible to rebound fracturing, especially in deep excavations. Long term material weathering can occur.



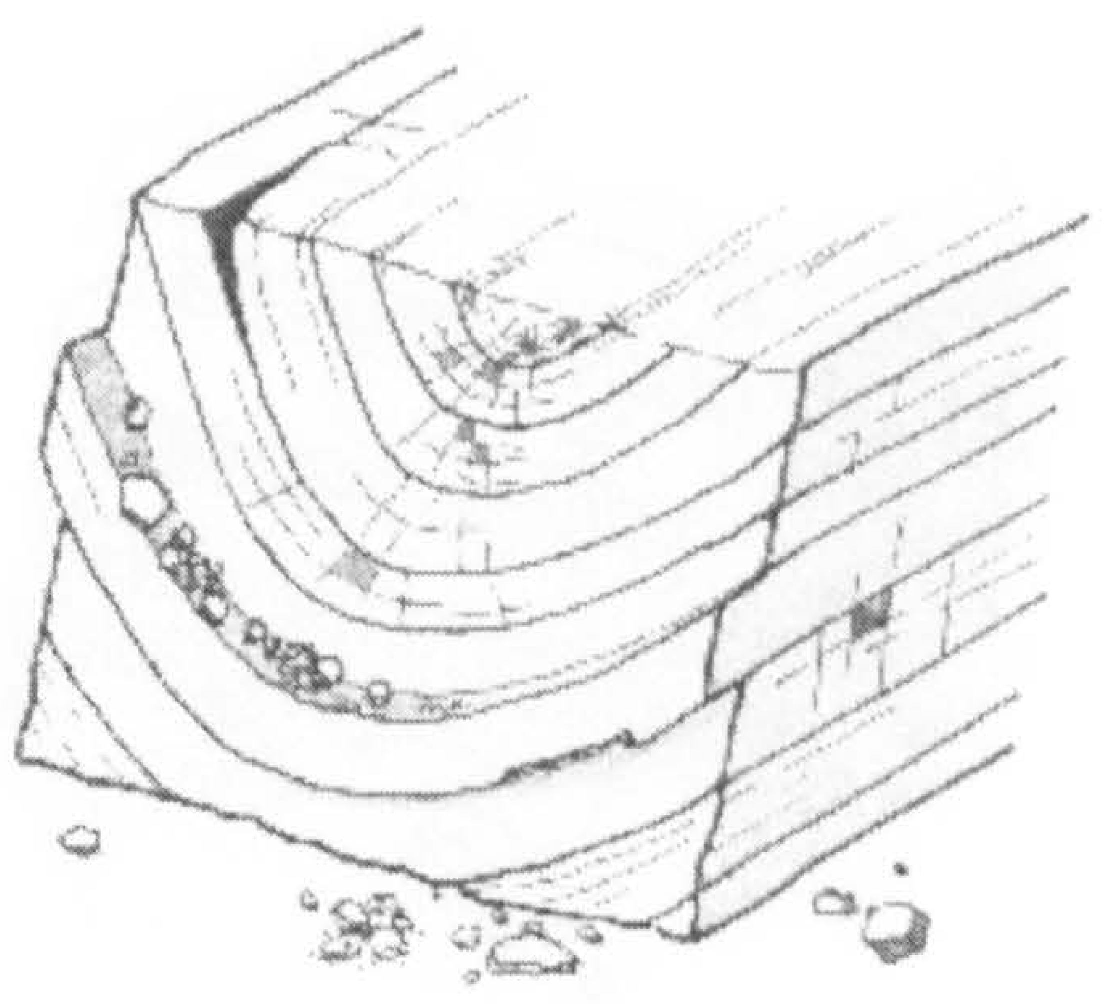
Associated deterioration modes: Wash erosion, contour scaling, stonefall, blockfall, (minor: *stone ravelling, flaking, slabfall, rockfall*)
Typical RDA_A Class: (Sed) **2-** (Ign) **1/2** (Met) **1+**.

Figure 8.10 Classification of rock mass types: ***Strong massive***

LAYERED

Description: Repeated layering of strata at any dip angle. Strata can be lithological (eg bedding) or structural (eg jointing). Prismatic (or columnar) structure is a sub-group (also for *regular blocky*), formed from the intersection of two persistent fracture sets. The two-dimensional trace of prismatic structure might appear as vertical layering.

Occurrence: Occurs in a wide range of bedded sedimentary rocks such as limestone, shale, sandstone and gritstone. Also in igneous rocks with repeated layering such as lavas (basalt, dolerite) and volcaniclastics. Occurs in foliated and banded metamorphic rocks such as slate, schist and some gneisses and metasediments. Can also occur in igneous rocks where a single set of closely spaced, regular joints is present.



Special characteristics: Horizontally layered masses can be very stable, but where folded or vertically layered, can be prone to erosion, flaking and collapse along chutes. Dip angle and direction control sliding of large blocks. Fall of isolated stones and blocks is the most common form of deterioration.

Associated deterioration modes: Stonefall, stone ravelling, wash erosion, blockfall (minor: *grain ravelling, flaking, solution, contour scaling, rockfall*).
Typical RDA$_A$ Class: (Sed) **2/3** (Ign) **2/3** (Met) **1 to 3**.

Figure 8.11 Classification of rock mass types: ***Layered***

FISSILE (LAYERED)

Description: Very thinly layered rock due to thin bedding, schistocity, cleavage or lamination. The term schistose structure can be applied where appropriate. Fissile structure tends to be dominated by regular, tight aperture discontinuity planes. Schistose structure is characterised by small scale, irregular, deformed foliation planes.

Occurrence: Occurs in clay or mica-rich rock including shale, very thinly bedded flaggy sandstone, cleaved metasediment, slate, schist and phyllite. Commonly occurs as a distinct zone in composite rock masses. Rare in igneous rock.

Special characteristics: Prone to material weathering and surface erosion, particularly where cyclic wetting and drying occurs. Usually produces large amounts of debris rapidly, producing significant slope regression in engineering time. More competent material can be classified as layered, and is often associated with slabfall and toppling deterioration modes.

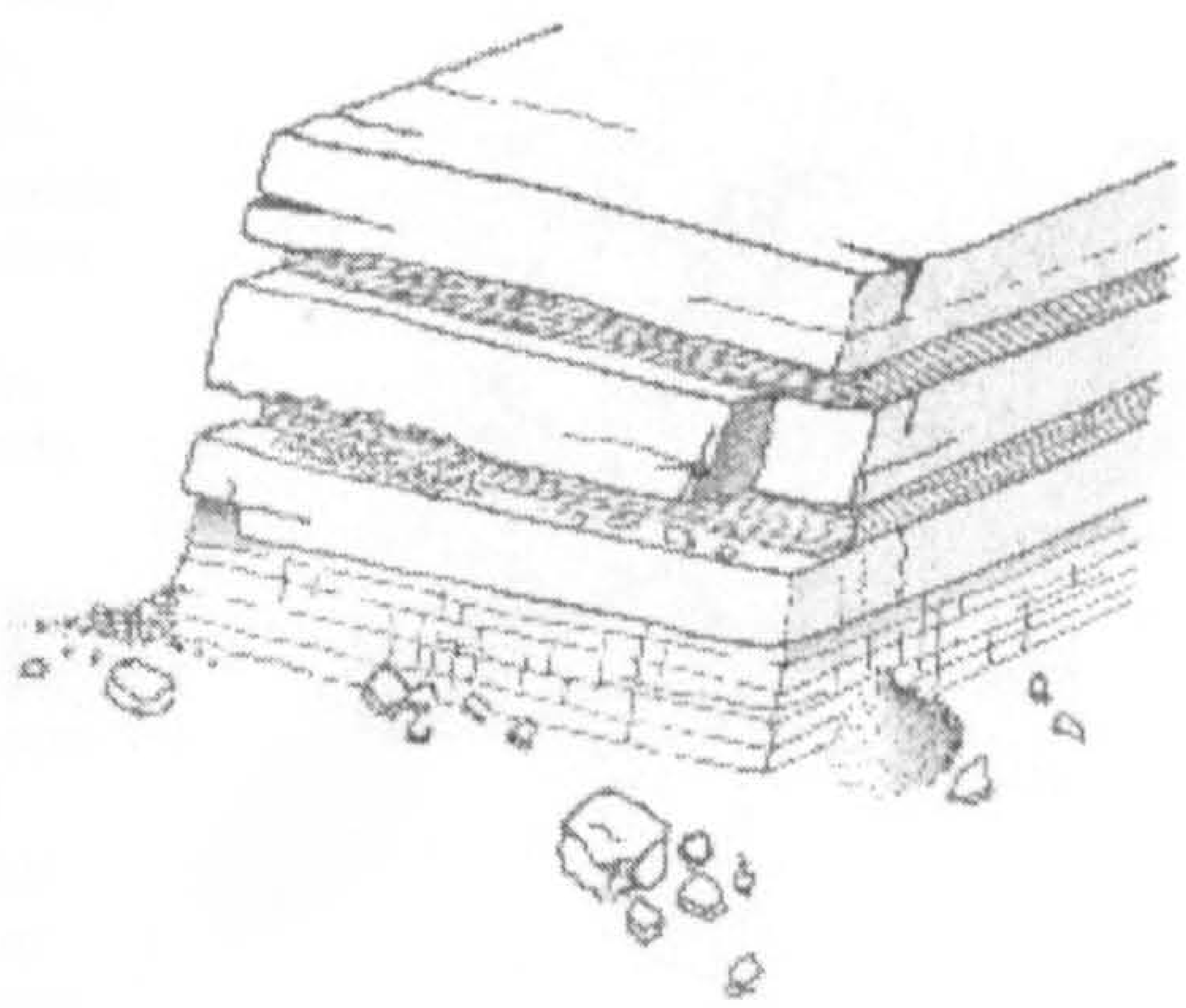
Associated deterioration modes: Flaking, grain ravelling, wash erosion (minor: *stone ravelling, stonefall, rockfall*)
Typical RDA$_A$ Class: (Sed) **4/5** (Met) **1 to 4**. Rare in igneous rock.

Figure 8.12 Classification of rock mass types: ***Fissile (layered)***

COMPOSITE (LAYERED)

Description: Inter-layered strata having contrasting material properties.

Occurrence: Occurs in sedimentary and metamorphic rock masses with inter-layered sequences of strata with contrasting properties. Examples include Coal Measures sequences of interbedded sandstone, shale and limestone; metamorphosed turbidite sequences; inter-bedded thin and thick layers of limestone. Can also occur locally at the site of unconformities. Can occur in interbedded lava sequences.



Special characteristics: Differential weathering of strata with contrasting properties leads to undermining, producing isolated and semi-continuous fall of stone sized material, and occasional collapse of overhangs. Protection or reinforcement of weaker strata essential to reduce undercutting and collapse of competent strata. Groundwater flow can be concentrated in the weaker, more porous layers.

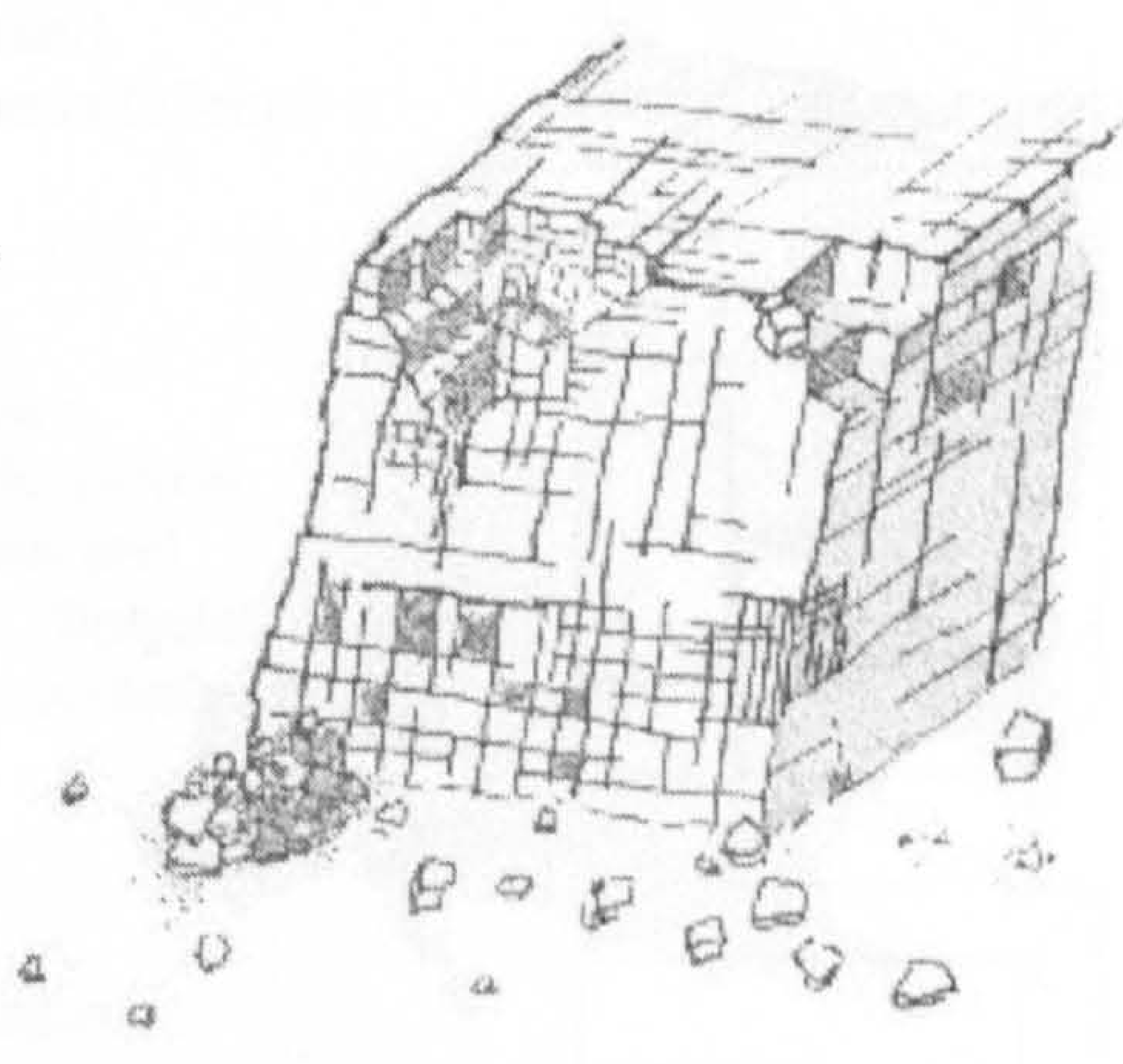
Associated deterioration modes: <u>Stonefall</u>, <u>stone ravelling</u> (minor: *block ravelling, flaking, wash erosion, contour scaling, rockfall*)
Typical RDA$_A$ Class: (Sed) **3+** (Met) **1/2**. Rare in igneous rock.

Figure 8.13 Classification of rock mass types: ***Composite (layered)***

REGULAR BLOCKY

Description: Orthogonal blocky structure due to intense and regular intersection of three or more sets of fractures. Prismatic (or columnar) structure is a rare sub-group (also for *layered*), formed from the intersection of two persistent fracture sets. The two-dimensional trace of prisms can appear as a regular blocky structure. *Rubbly* can be used as a suffix term for blocky structure in weak chalk and oolitic limestone.

Occurrence: Occurs in a wide range of moderately strong to strong sedimentary rocks including gritstone, strong sandstone, crystalline limestone and hard chalk. Occurs also in igneous rocks which have well developed jointing structure (eg from cooling contraction) such as granite, basalt, pegmatite and microgranite. Occurs in strong metamorphic rocks including gneiss and metasediments. Also common in folded and faulted rock masses.



Special characteristics: Most deterioration relates to rock mass properties and material weathering is usually incidental. Fracturing often enhanced by stress release jointing. Blocky rock masses are commonly associated with high frequency falls.

Associated deterioration modes: <u>Stone ravelling</u>, stonefall, blockfall, rockfall (minor: *grain ravelling, flaking, wash erosion, contour scaling*)
Typical RDA$_A$ Class: (Sed) **3+** (Ign) **2** (Met) **2/3**.

Figure 8.14 Classification of rock mass types: ***Regular blocky***

IRREGULAR BLOCKY

Description: Irregularly shaped and variably sized blocks due to non-patterned intersection of fractures. Rubbly should be used as a suffix term for irregular blocky structure in weak chalk and oolitic limestone.

Occurrence: Occurs in rocks where shattering from stress release, blasting and weathering dominates structure. Found in a wide range of moderately strong to strong sedimentary rocks including gritstone, strong sandstone, crystalline limestone and hard chalk. Also occurs in strong metamorphic rocks including gneiss and metasediments. Only occurs in igneous rocks where regular jointing is absent. These include pillow lavas, ignimbrite, tuff and some microgranites. Also occurs in weak chalk, particularly in association with nodular beds.

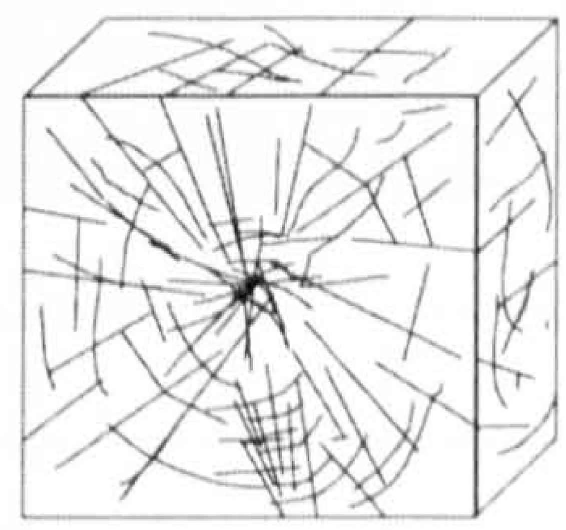
Special characteristics: Irregular fracturing associated with intensely folded and faulted rock, curved cooling or sheeting joints, shear zones, stress release, blasting and weathering. In strong rock masses, irregular shape of blocks causes tight interlocking, reducing block release potential. Deterioration tends to involve large blocks, or large volumes of material. Although rare, debris flow can occur in this rock mass type.

Associated deterioration modes: Stone ravelling, stonefall, rockfall (minor: *grain ravelling, flaking, wash erosion, blockfall, debris flow*)
Typical RDA_A Class: (Sed) **2 to 4** (Ign) **2/3** (Met) **1 to 3**.

Figure 8.15 Classification of rock mass types: ***Irregular blocky***

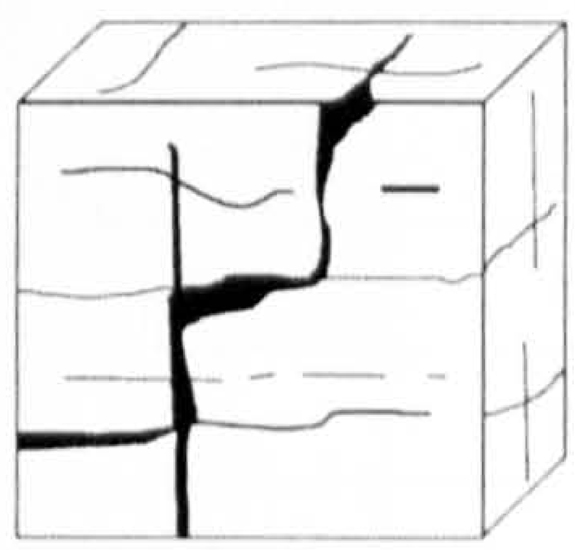
SUBSIDIARY ROCK MASS STRUCTURES

INTENSELY FRACTURED ZONES



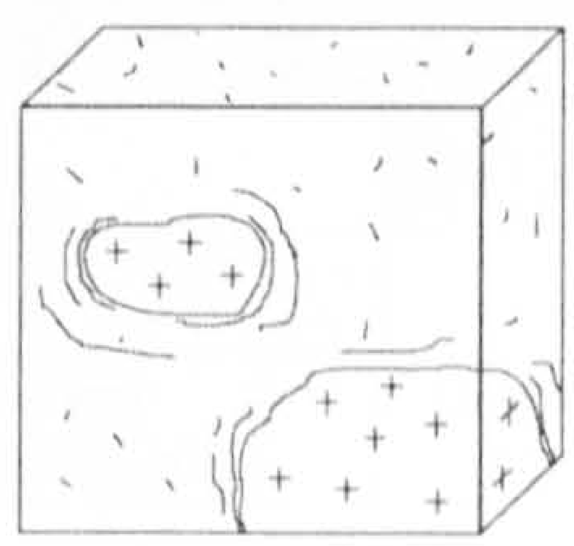
Description: Intense, localised fracturing and shattering, producing a very loose structure. **Occurrence:** All rock types. Usually associated with blast damage zones, plant roots, the hinges of tightly folded strata, faults and shear zones. **Special characteristics:** Vulnerable to root penetration, increased groundwater flow and direct disturbance. Commonly leads to local but intense ravelling, rockfall and debris flow.

SOLUBLE ROCK MASSES



Description: Solution of rock material, leading to fracture enlargement, micro-solution features and macro karstification in severe cases. **Occurrence:** Soluble rocks (eg crystalline and oolitic limestone, chalk), and rocks with soluble cement. **Special characteristics:** Collapse of solution cavities leads to rockfall and blockfall.

COMPOSITE STRUCTURE



Description: Two or more types of contrasting rock mass structure are present in close proximity, in a non-layered, non-uniform distribution. **Occurrence:** Examples include profiles with corestones, igneous intrusions, buried channels and exposure of palaeo-weathering profiles. **Special characteristics:** Might lead to overhang collapse where differential weathering occurs.

Figure 8.16 Classification of rock mass types: ***Subsidiary rock mass types***

8.5 An Engineering Classification of Deterioration Modes

An outline classification of deterioration modes was given in Chapter Seven (section 7.3.4). A series of detailed data sheets is presented here (Figure 8.18 to 8.28), giving a brief description of the mode, its typical geological occurrence, geotechnical implications, and an indication of appropriate mitigation and maintenance measures. For proposed (or new) slopes, estimation of the likely rock mass type will enable preliminary assessment of deterioration modes most likely to occur. For existing, older slopes, deterioration morphology can assist evaluation of deterioration modes (section 8.6). Figure 8.17 provides a summary of the likelihood of each deterioration mode occurring in each major type of rock mass and is based on data presented in Chapter Seven (Figures 7.17a to g) and Appendix 7.A.

8.5.1 Influence of deterioration mode on treatment selection

As stated earlier, one of the key reasons for identifying and classifying deterioration modes is because they each have different implications for deterioration severity, the consequences of deterioration, and selection of appropriate mitigation and maintenance measures. This is reflected in the treatment measures matrix presented in Stage Three of RDA (Table 8.3).

	Weak Massive	Strong Massive	Layered	Fissile (Layered)	Composite (Layered)	Regular Blocky	Irregular Blocky
Grain ravelling	5	1	2	3		2	2
Stone ravelling	1	2	4	2	4	5	4
Block ravelling			1	1	2	1	1
Flaking		2	2	5	2	2	2
Wash erosion	3	4	3	3	2	2	2
Solution and karstification		1	2			1	
Flexural toppling				1			
Grainfall	5	1	1				
Stonefall	2	4	4	2	4	3	3
Blockfall		3	3	1		3	2
Contour scaling	3	4	2	1	2	2	1
Slabfall		2	1		1	1	1
Toppling		1	1			1	
Rockfall	1	2	2	2	2	3	3
Debris flow							2
Rockslide			1			1	

5	Extremely likely to occur	4	Very likely to occur	3	Likely to occur	2	Might occur	1	Local influence only		Unlikely to occur

Figure 8.17 Summary of deterioration modes in relation to rock mass type

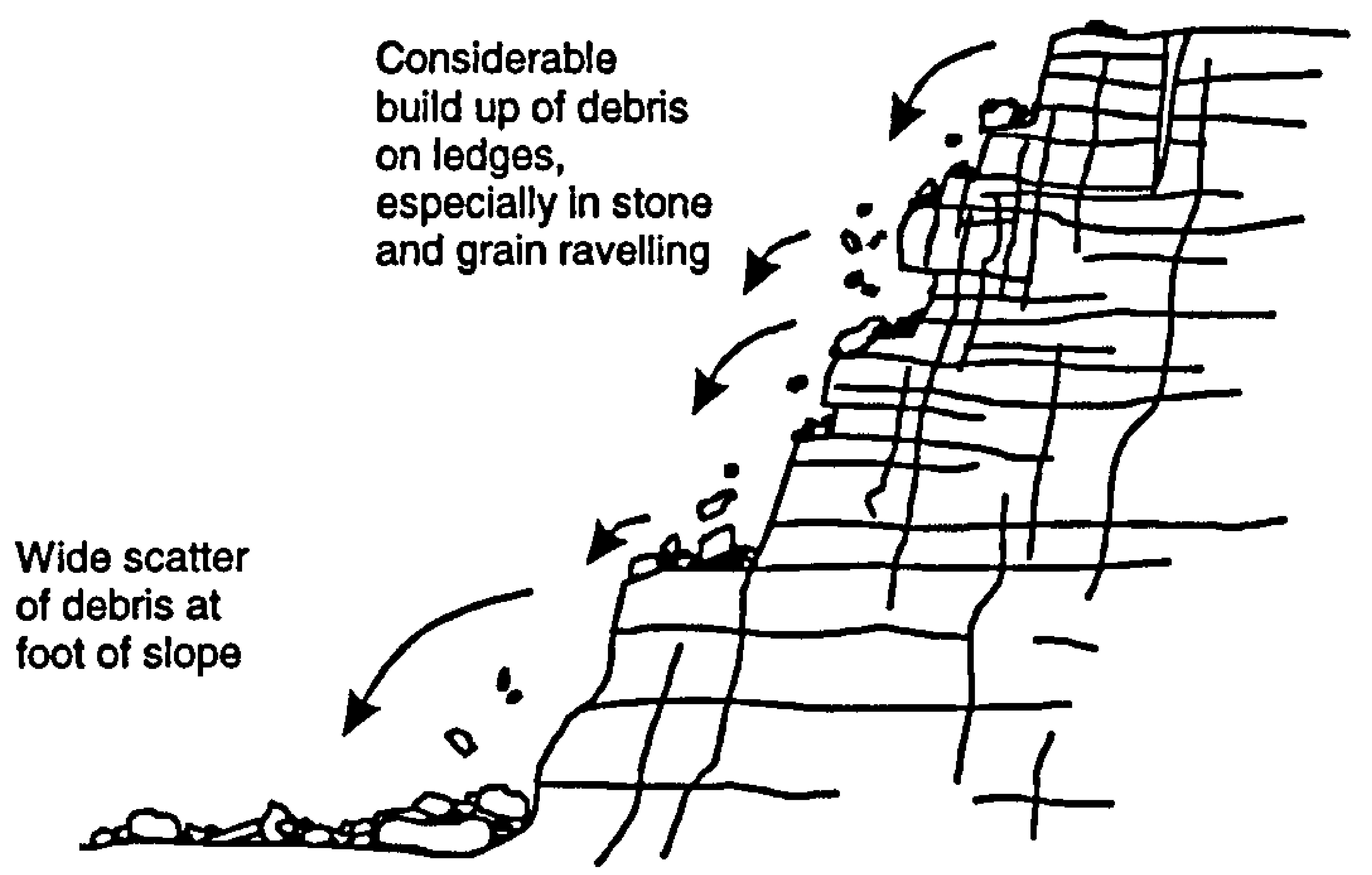
RAVELLING

Ravelling is the frequent and semi-continuous fall of material of any dimension. The fall of individual particles due to ravelling is difficult to predict in time or space, though weakened or intensely fractured zones which might be prone to this mechanism can usually be identified. Three sub-types are recognised, grain, stone and block ravelling.

Occurrence: *Grain ravelling* occurs in weak, granular and soil-like rock masses which are prone to grain detachment. It can be transitional with wash erosion (Figure 8.20), and can be enhanced by raindrop impact or wind erosion. *Stone ravelling* can also be transitional with wash erosion and occurs in moderately strong, intensely fractured rock masses. *Block ravelling* occurs in strong, fractured rock masses such as layered sandstone or blocky limestone.

Geotechnical implications:

Grain ravelling: The direct consequences of grain ravelling are minimal. *Stone ravelling:* Debris at the slope foot is often widely scattered, particularly if stones bounce on descent. *Block ravelling:* produces a wide scatter of debris at the slope foot. Blocks can land at some considerable distance from the slope, particularly if they bounce or roll during descent or disaggregate on impact.



Schematic illustration of ravelling

Mitigation and maintenance:

Grain ravelling: Periodic clearance of debris from clogged drains might be required. Surface treatment with shotcrete or geotextile membranes such as coir netting might be appropriate. In some cases, vegetation cover might assist in reinforcement of rock material.

Stone ravelling: Regular inspection and removal of loose stones and clearance of debris might be necessary. Where there is an unacceptable risk, loose material can be retained cost-effectively using wire mesh netting. Shotcrete is also effective for where the constituent material is small. Rocktrap ditches are also very effective in containing debris although periodic clearance might be necessary. Other active mitigation measures include protective fencing and buttressing in more severe cases.

Block ravelling: For unprotected slopes where there is some risk, regular inspection is required, as is the removal of loose blocks and clearance of debris. Falls of material can be contained with a rocktrap ditch and fence, and intermediate rocktrap fences on benches will limit throw due to bouncing. Blocks which fall to the toe can be retained in a catch ditch which might need to be cleared out regularly. Larger blocks can be secured individually with grouted dowels or rockbolts, but areas prone to severe block ravelling might require full support (eg local buttressing), containment (eg substantial revetment or gabion walling), and protection (eg rockfall shelters and warning signs for severe cases).

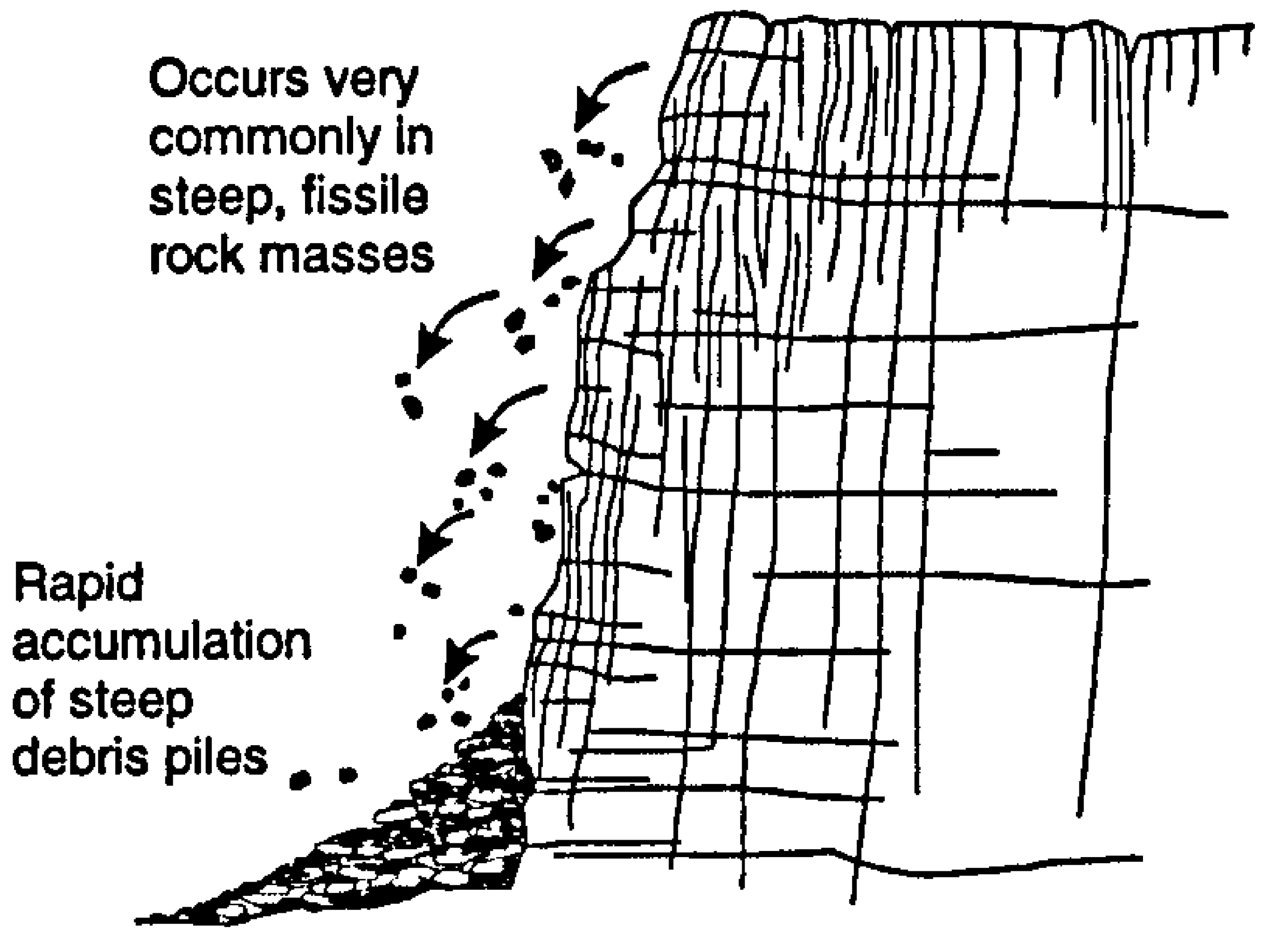
Figure 8.18 Classification of deterioration modes: ***Ravelling***

FLAKING

Flaking is a special form of ravelling, involving frequent and semi-continuous fall of material with a distinct platy form.

Occurrence:
Flaking occurs in very fissile rocks like shales and slates

Geotechnical implications:
Flaking can rapidly produce steep, extensive debris piles at the slope foot. Debris is unlikely to spread far, but build up commonly occurs so rapidly that very frequent foot clearance is needed. Flaking can result from repeated wetting and drying.



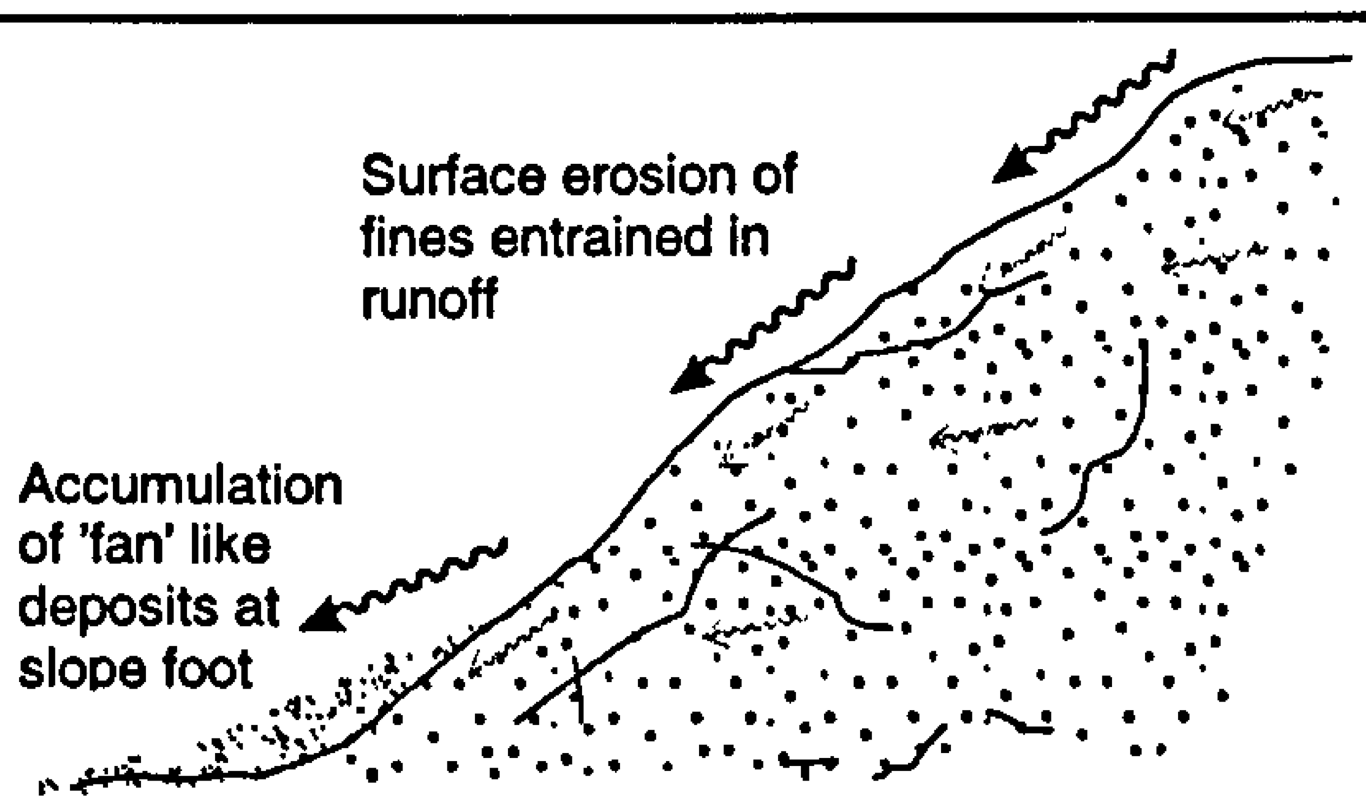
Schematic illustration of flaking

Mitigation and maintenance:
Retention of individual flakes is clearly impractical, and deep penetration of weathering often precludes the removal of loose material. A cover of fine wire mesh or geotextile membrane, however, might be effective although any covering will need to be pinned to the slope. Standoff area at the slope foot can be increased to provide more fallout space for debris, or rocktrap ditches used to contain it. In particularly weathered material, vegetation cover can be effective.

Figure 8.19 Classification of deterioration modes: ***Flaking***

WASH EROSION

Wash erosion involves the detachment and transport of fine material entrained in surface water runoff.



Schematic illustration of wash erosion

Geotechnical implications:
Debris deposition on the slope is usually widespread and includes a large proportion of fines. At the foot of the slope this can cause nuisance by clogging drains. Erosion due to channelled surface runoff can lead to rill and gully development. Severe gullying will change the slope geometry.

Occurrence:
Wash erosion occurs in weak, soil-like rock masses (eg weathered sandstone and mudstone), commonly on low angle slopes. It occurs locally in highly fractured rock masses where constituent materials are variably weathered. Wash erosion is very commonly the major deterioration mode operating on strong massive slopes although on such slopes it usually has less severe consequences.

Mitigation and maintenance:
A key factor in remedial treatment is the removal of the primary source of erosion by installing drainage, the extent of which will relate to the potential severity of deterioration. Crest and slope drainage are essential and trench drains can be used in severe cases. Toe drainage also helps to reduce scour. Benches or other cross flowpath barriers can be constructed in severe cases to reduce downslope flow velocity, and in the severest of cases, slope angle and length can be reduced. Slope surface protection using geotextiles and vegetation is usually very effective in reducing runoff velocity and hence erosive power, and in severe cases rockfill mattresses can be used as surface cover (eg cribbing or gabions). Localised shotcrete application (with drainage holes) might also be helpful.

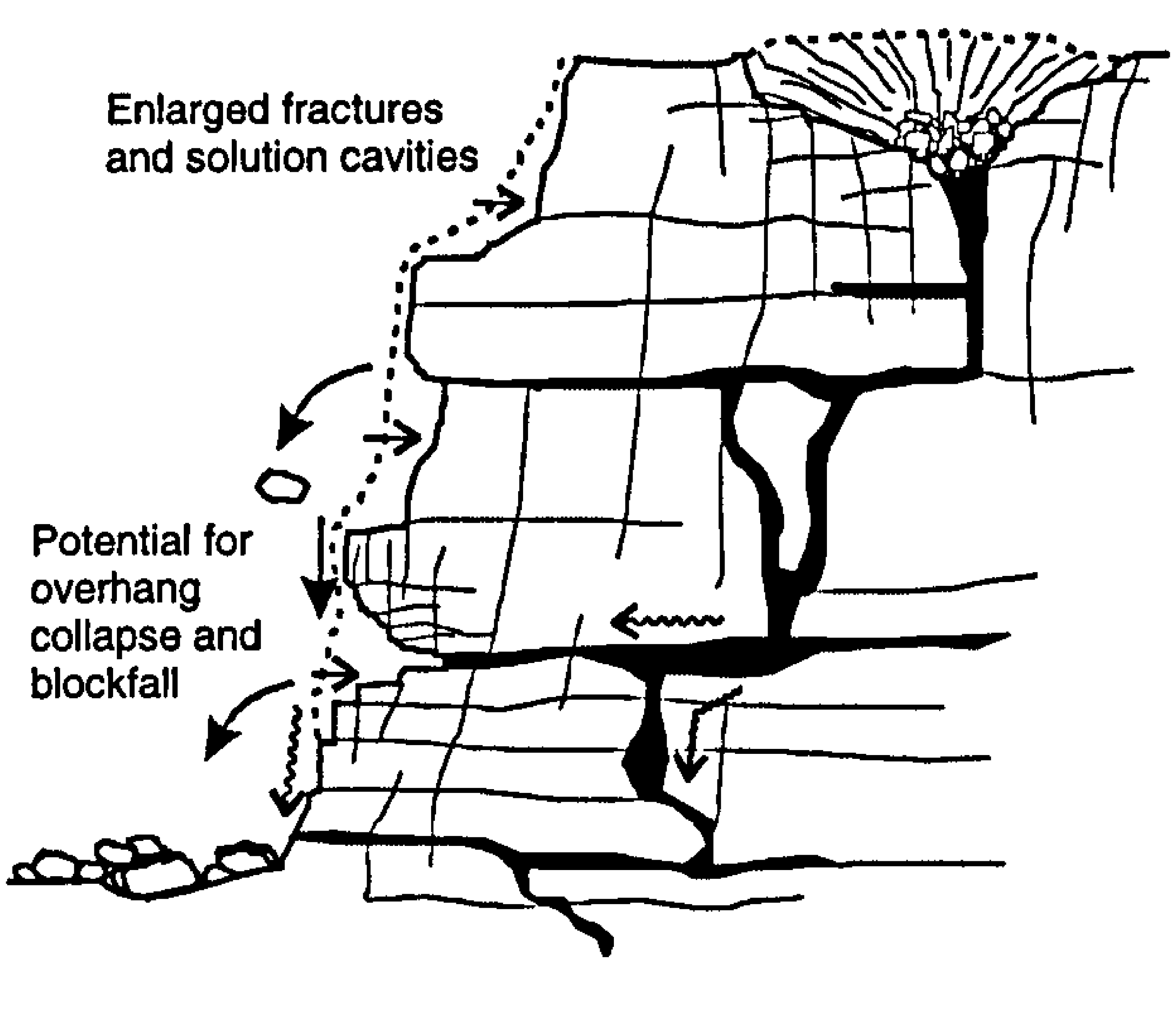
Figure 8.20 Classification of deterioration modes: ***Wash erosion***

SOLUTION AND KARSTIFICATION

Solution and karstification involve the dissolution of soluble mineral grains and cementing material in aggressive, acid solutions, including rainwater.

Occurrence: *Solution* can develop karst forms in some soluble rocks, particularly limestone, though this is less common in chalk. Karstification occurs rapidly in gypsum, though is rarely encountered in excavated rockslopes. Solution produces micro-solution forms in intact rock, and can lead to fractures enlargement (especially in vertical fractures).

Geotechnical implications:
Solution cavities can develop and are prone to collapse. Palaeokarst forms might also affect rock mass structure and geometry where exposed.



Schematic illustration of solution and karstification

Mitigation and maintenance:
Large cavities can be underpinned or otherwise supported and smaller cavities can be infilled with mortar screeding. Water ingress can be reduced by crest and slope drainage, together with sealing and draining of vertical fractures at the rear of the slope.

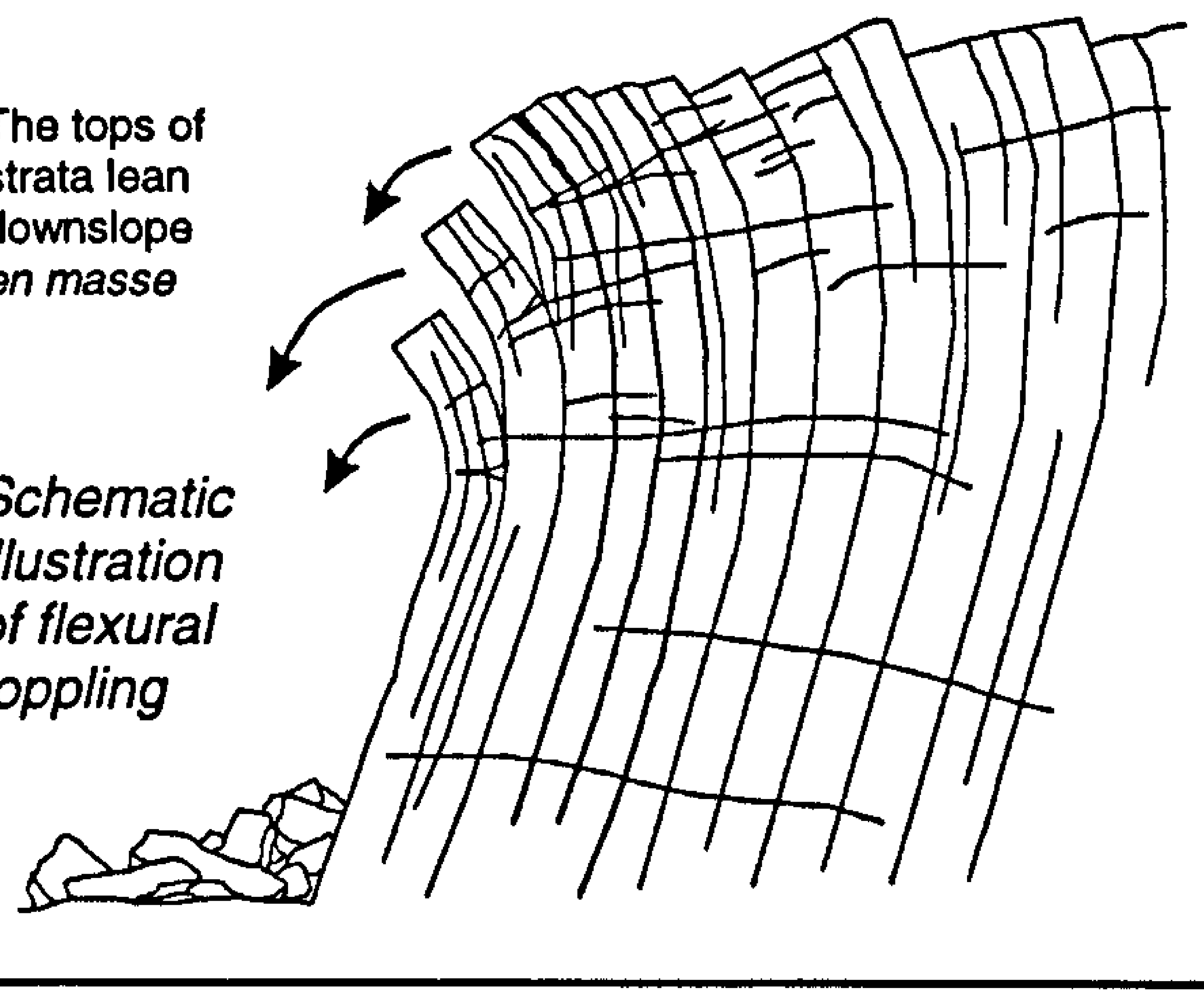
Figure 8.21 Classification of deterioration modes: ***Solution and karstification***

FLEXURAL TOPPLING

Flexural toppling is a slow, progressive deformation and sliding of layered strata due to gravitational forces upon removal of lateral constraint.

Occurrence: Flexural toppling commonly occurs in fissile, thinly bedded and composite rock masses in steeply dipping strata, the exposed tops of which tend to bend downslope.

Geotechnical implications: While flexural toppling in itself presents little hazard, it frequently leads either to large scale failure of the rock mass by rockfall or to semi-continuous failure by toppling and ravelling.



Schematic illustration of flexural toppling

Mitigation and maintenance:

Long term movement monitoring is essential in the remediation process. It might be possible to seal and drain major vertical fractures at the rear of the slope to reduce water pressure. Rock anchors can be used to gain substantial support. Identifying key blocks for individual reinforcement is important.

Figure 8.22 Classification of deterioration modes: ***Flexural toppling***

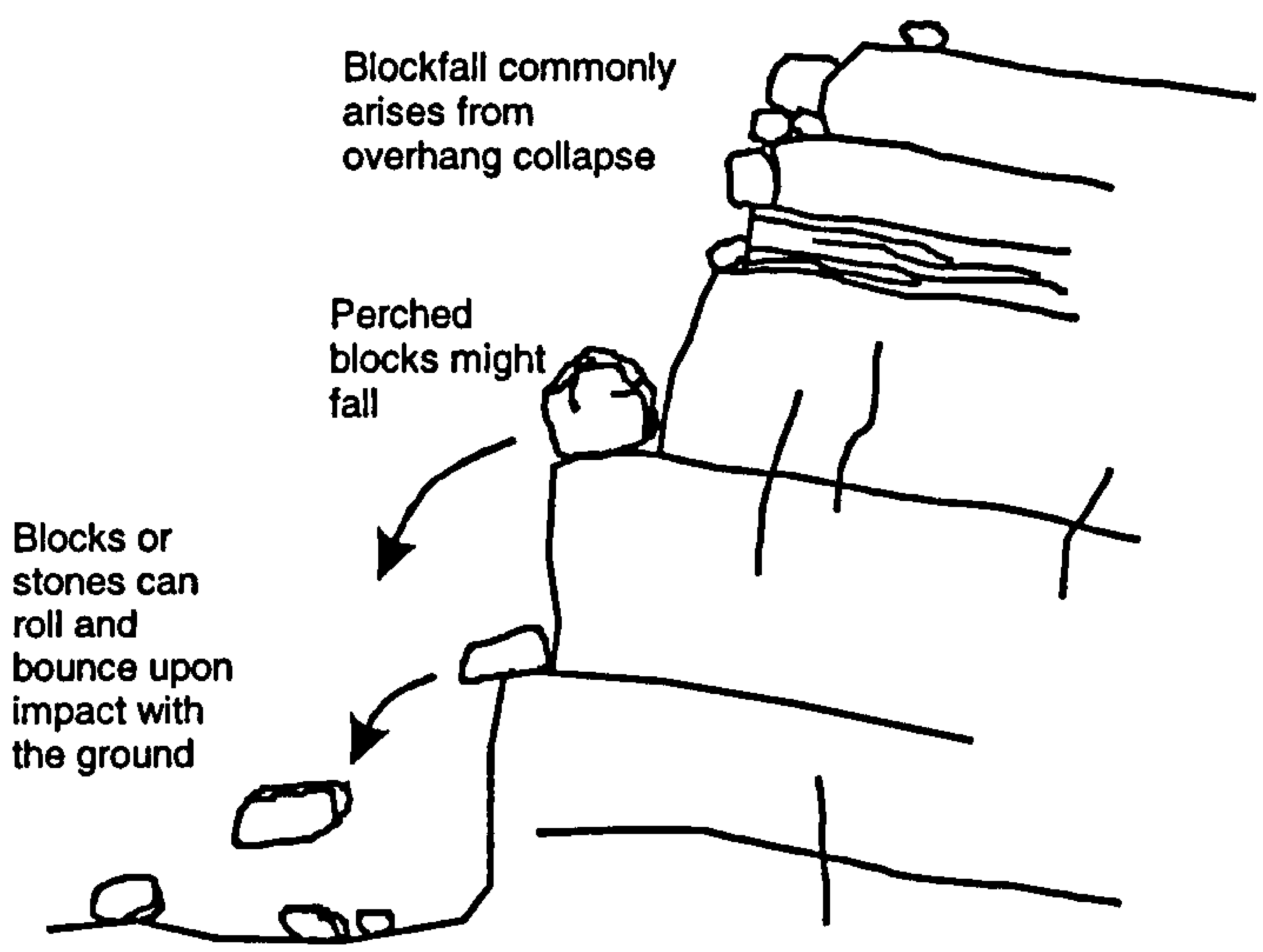
FALL

Fall describes the occasional release of individual rock fragments. The larger the particle size, the easier potential falls are to identify and treat. Many isolated falls occur in previously displaced material which accumulates on ledges. Three sub-types of fall, grainfall, stonefall and blockfall are considered separately below.

Occurrence: *Grainfall* occurs in weak massive rock masses with a granular texture. *Stone fall* occurs in stronger rock masses with occasional, localised loose zones. *Blockfall* occurs in strong, blocky, layered and composite layered rock masses with widely spaced fractures, and is most commonly associated with freefall from small overhangs or areas affected by root wedging from woody vegetation.

Geotechnical implications:

Grainfall: This mechanism presents very little hazard. *Stonefall:* Bouncing and rolling of detached stones increases the spread of debris, and can present a serious risk (to road users, for instance). *Blockfall:* Bouncing and rolling of detached blocks and their disaggregation on impact increases the debris spread and presents a serious hazard. The fall of large blocks can also cause direct impact damage to structures such as toe drains and pavement edges.



Schematic illustration of fall mechanisms

Mitigation and maintenance:

Grainfall: Treatment is rarely necessary. This can be confirmed by infrequent inspection.

Stonefall: Regular slope inspection is necessary to identify vulnerable stones for removal. Masonry dentition or shotcrete can be applied to retain loose material, and small overhangs can be supported. Use of rocktrap fencing will largely remove the risk in many cases.

Blockfall: Regular slope inspection coupled with movement monitoring is necessary to identify vulnerable blocks, which can be secured with bolts, dowels, and cables where inaccessible. Loose blocks can be retained with masonry dentition and underpinning of overhangs. Rocktrap ditches reduce debris spread at the foot of the slope and fencing can counteract high bouncing.

Figure 8.23 Classification of deterioration modes: ***Fall***

CONTOUR SCALING

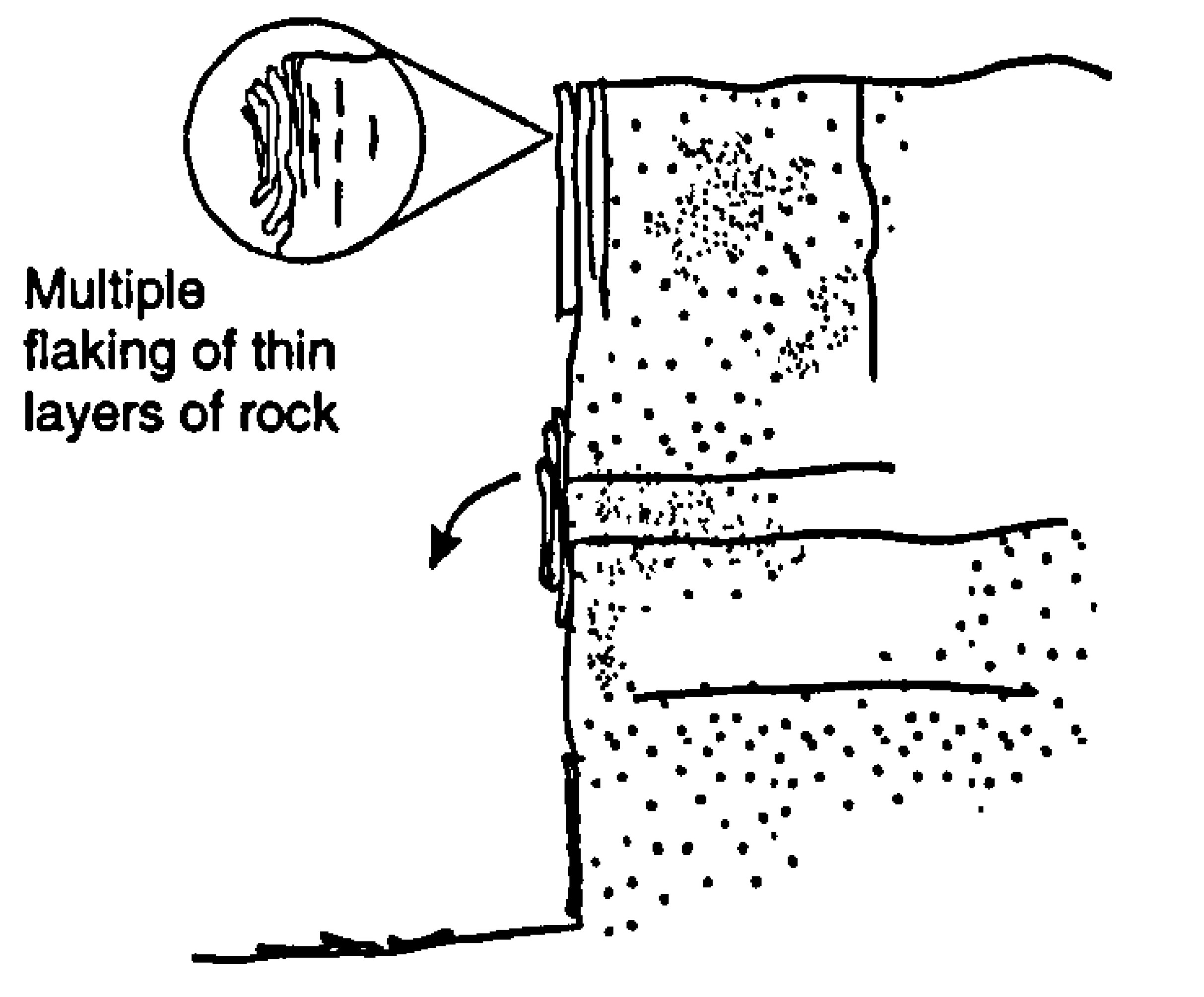
Contour scaling is a special form of fall involving the occasional exfoliation of thin layers of rock material formed parallel to the slope surface. The thickness of layers, usually several millimetres, often corresponds with the depth to which material weathering has penetrated.

Occurrence:

Contour scaling occurs most commonly in moderately strong, massive rocks such as some sandstones and chalks.

Geotechnical implications:

Large scales can disaggregate on impact causing limited debris spread at foot, but the risk is small.



Schematic illustration of contour scaling

Mitigation and maintenance:

Loose material can be removed periodically, though selective application of shotcrete might also be effective.

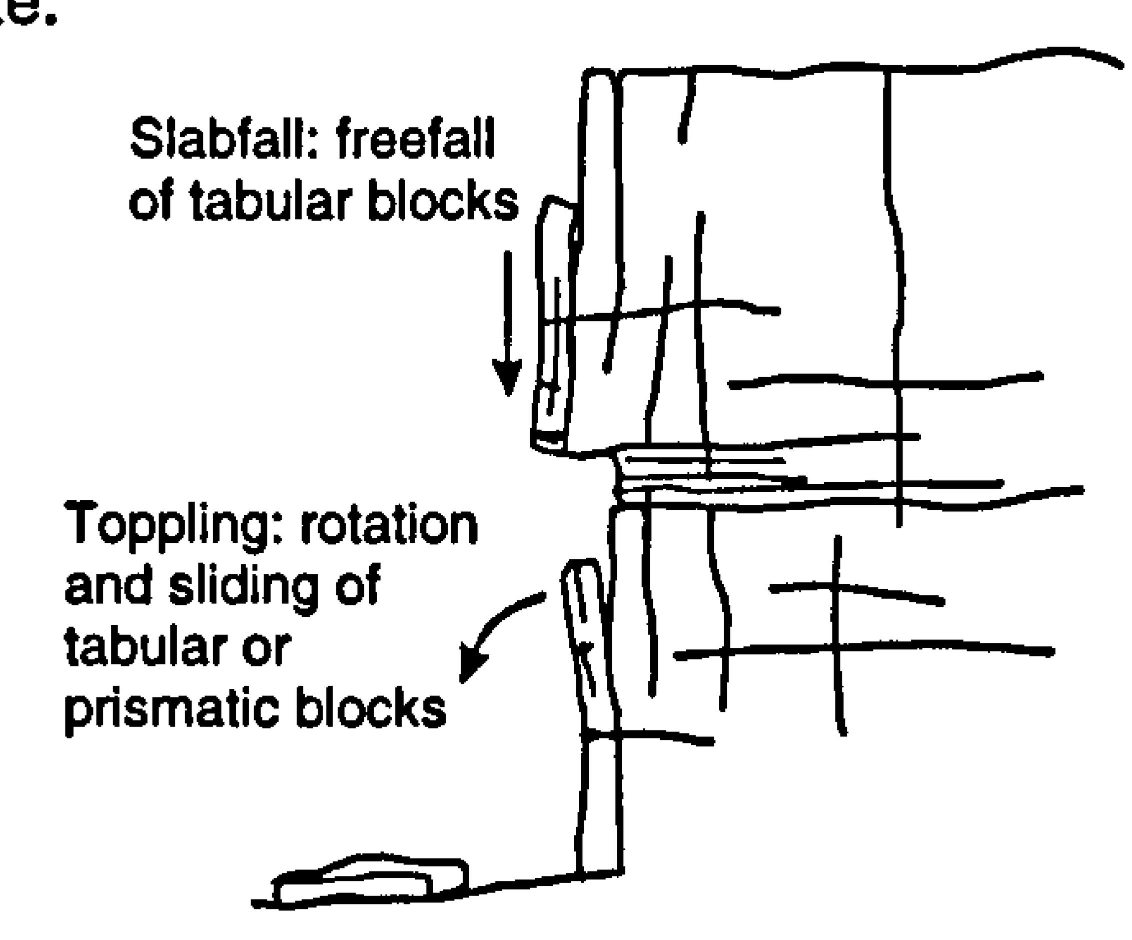
Figure 8.24 Classification of deterioration modes: ***Contour scaling***

SLABFALL AND TOPPLING

Slabfall and toppling are also forms of fall, involving isolated and infrequent freefall of large, tabular slabs and rotation of large prismatic blocks. A typical minimum 'a' axis dimension of such slabs and blocks is one metre. Material of smaller dimensions which fails in this way can be described as stonefall or blockfall as appropriate.

Occurrence: *Slabfall and toppling* occur in stronger rock masses with discontinuities parallel to a steep slope plane.

Geotechnical implications: Because of the large material size involved, there is a significant risk. Slabs and topples can damage drainage channels, fencing and paving by direct impact, though debris is unlikely to spread far unless disaggregation on impact or bouncing on ledges occurs. It is common for overhangs and unstable areas to be left behind on the slope after slabfall or toppling has occurred.

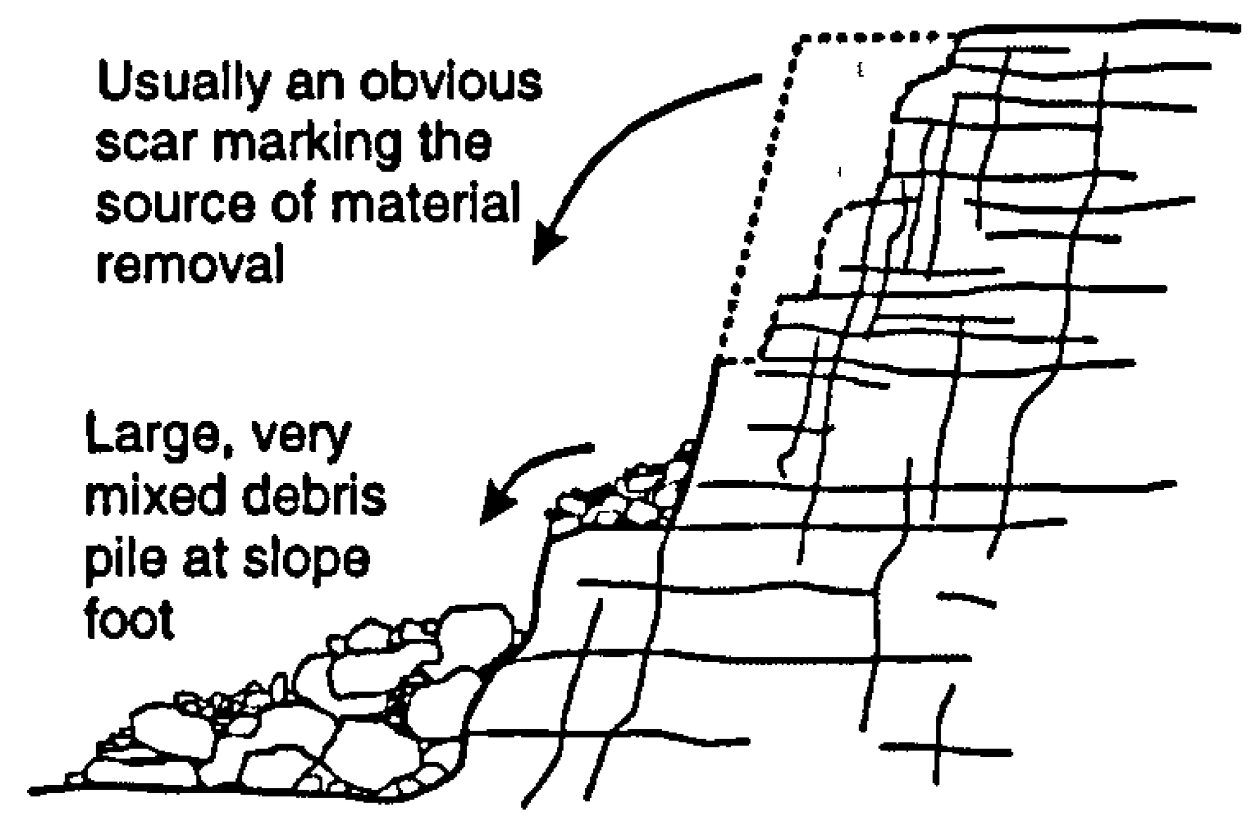


Schematic illustration of

Mitigation and maintenance:
Individual vulnerable blocks are generally easy to identify, and can either be removed or retained by bolts or cables. Overhanging slabs can be supported by underpinning, and severe cases require construction of substantial support structures. Depending on the scale of the potential hazard, rocktrap ditches and fencing might be inadequate to mitigate risk.

Figure 8.25 Classification of deterioration modes: ***Slabfall and toppling***

ROCKFALL



Schematic illustration of rockfall

Rockfall is used here as a specific term to describe the fall of many blocks of varying sizes in a single, identifiable event, and involves slide, roll, bounce and freefall. *Overhang collapse* is a specific form of rockfall, usually involving vertical freefall only.

Occurrence: *Rockfall* occurs in fractured and weakened rock masses on steep slopes where lateral and/or vertical support has been removed (eg due to undercutting, erosion and weathering), but is relatively infrequent. The volume of material involved might be dictated by the presence of a shallow, but irregular failure plane, though many rockfalls are associated with collapse of overhangs and cavities.

Geotechnical Implications: *Rockfalls* can result in the spread of a considerable amount of debris at the foot in a way which is difficult to predict. The potential consequences on a road can be severe.

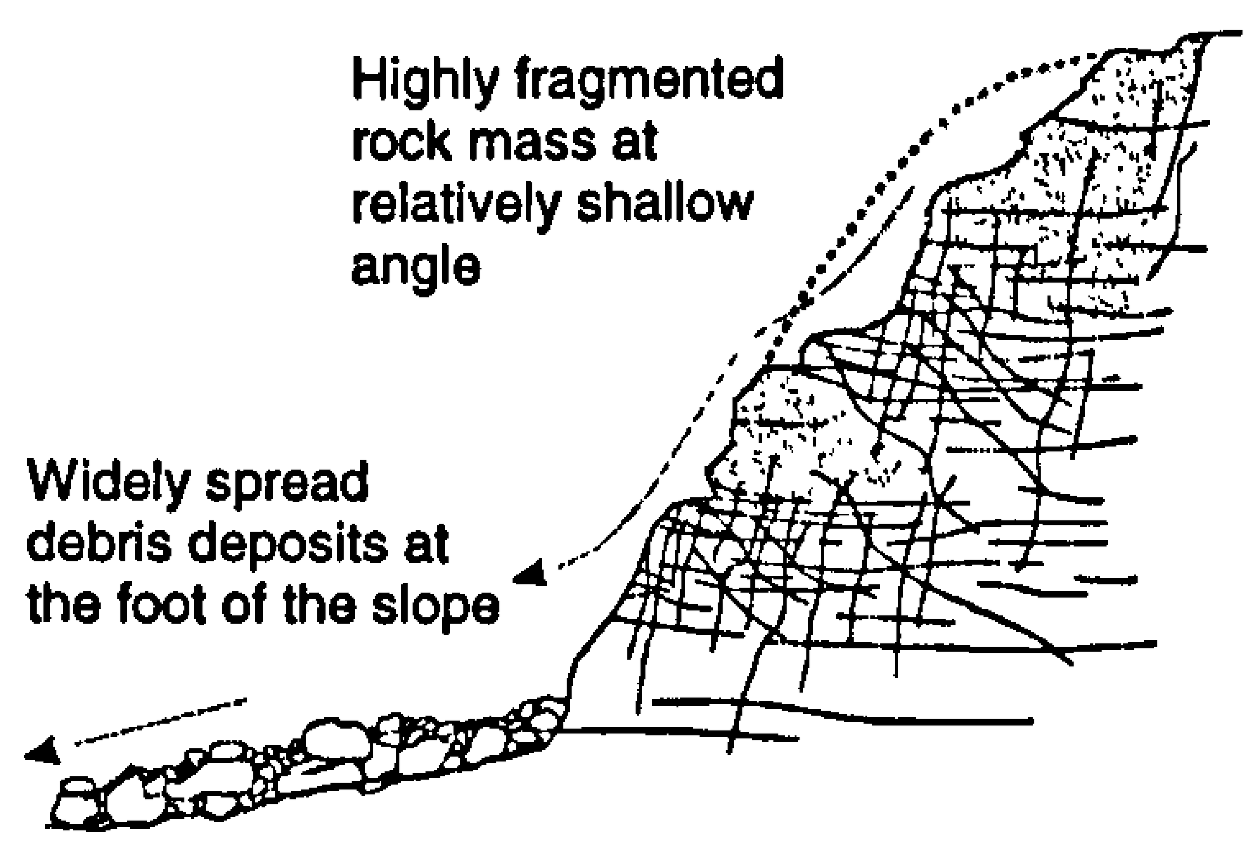
Mitigation and maintenance:
If the cause of weakening can be identified it should be treated or removed, and the slope monitored. Scaling back to a failure plane is problematic if there is potential for retrogression, and containment using wire mesh, shotcrete, or dentition is only useful if the potential rockfall is small. Local underpinning of medium potential falls, and full support of large potential falls is usually required (eg substantial revetment, underpinning with anchored, reinforced concrete beams, buttressing, gabion or crib walling). Other measures include rockfall shelters, protective walling and warning signs.

Figure 8.26 Classification of deterioration modes: ***Rockfall***

DEBRIS FLOW

Debris flow is the rapid transport of a mixture of coarse and fine particles in a partially saturated, grain-supported flow, and involves initial sliding and subsequent flow processes.

Occurrence: *Debris flow* occurs in highly fragmented, weathered and soil-like rock masses, usually at relatively shallow slope angles (<60°). They are rare and are often transitional with rockfall.



Schematic illustration of debris flow

Geotechnical implications: The potential for debris flow is difficult to identify, though. Debris flows often result in large depositional lobes with extensive spread at the foot due to channelisation and entrainment of debris.

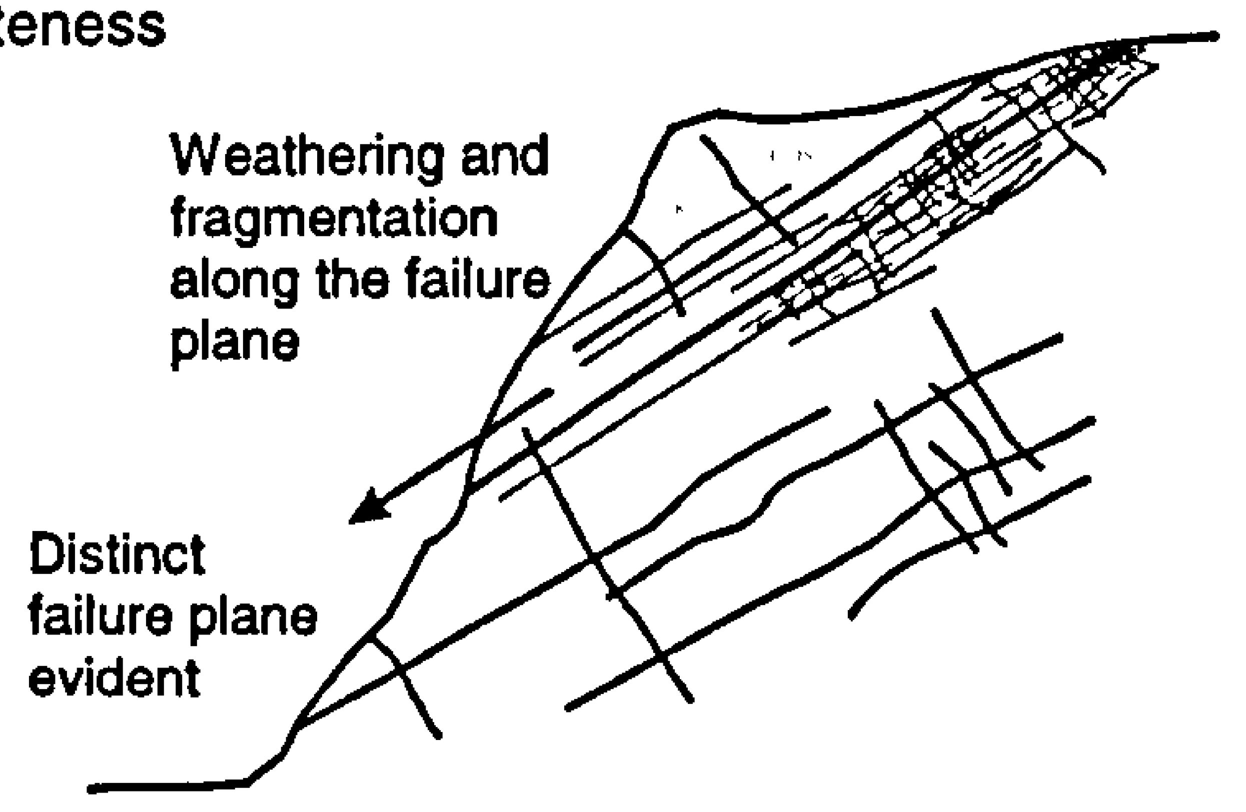
Mitigation and maintenance: Small scale flows (eg like wash erosion) can be mitigated with shotcrete and local dentition. Rocktrap ditches and fences can quickly become overwhelmed. The most effective measures involve complete support at source with use of revetments, buttressing, gabion or crib walling. Crest and slope drainage is also essential to reduce infiltration and other active drainage measures should be considered. Where stabilisation at source is not possible and the consequences cannot be mitigated by moving structures out of the potential pathway, cross flowpath barriers can be constructed and the slope angle reduced.

Figure 8.27 Classification of deterioration modes: ***Debris flow***

ROCKSLIDE

Rockslides are rare, relatively large scale rapid translational movements of intact rock along a distinct, planar discontinuity. Where the material is fragmented or grain-supported, the term debris slide or flow should be used. Rockslides commonly degenerate downslope into debris flows as intact material is disaggregated. Large rockslides might constitute quantifiable slope failures and can be analysed by limit equilibrium methods, so are strictly outside the scope of this research - they are included here only for completeness and because small scale, shallow forms can occur.

Occurrence: *Rockslides* occur where discontinuities strike roughly parallel to the slope plane and dip at a relatively steep angle for the available shear strength. They can be triggered by progressive pre-failure weathering along the discontinuity and it might be possible to treat this weathering at source to limit weakening. Failure is often preceded by relatively minor movements. Infilled or open vertical fractures behind the slope can also be an important indicator of potential failure.



Schematic illustration of rockslide

Geotechnical Implications: Large *rockslides* have massive potential for damage, destruction and loss of life, chiefly due to the volume of material involved and the associated extensive spread of debris. Small rockslides have similar implications as for rockfalls and debris flows.

Mitigation and maintenance: Large potential rockslides should be regarded as major slope instability and treated appropriately (eg anchor reinforcement, toe weights, access restriction and substantial drainage). Small volume, shallow rockslides will benefit most from movement monitoring, drainage and local bolting or other support/reinforcement.

Figure 8.28 Classification of deterioration modes: ***Rockslide***

8.6 Deterioration Morphology

Plates 8.1 to 8.8 provide photographic examples of some of the erosional, depositional and in situ landforms derived from deterioration as described in Chapter Seven. Further information is given on their occurrence and the deterioration modes associated with them. This can be used to assist in the interpretation of deterioration active on *existing* excavated rockslopes.

8.7 The Timing of Deterioration

Weathering and erosion processes vary in response to diurnal, seasonal, annual and secular fluctuations in many of the factors which influence and control deterioration. The mechanisms of deterioration are not understood clearly enough for accurate predictions of their timing to be made. Nevertheless, a general appreciation of temporal variations in deterioration can be useful in the planning and design of maintenance operations and slope stabilisation and protective works. Some of these variations and their effects with rockslope deterioration are considered below.

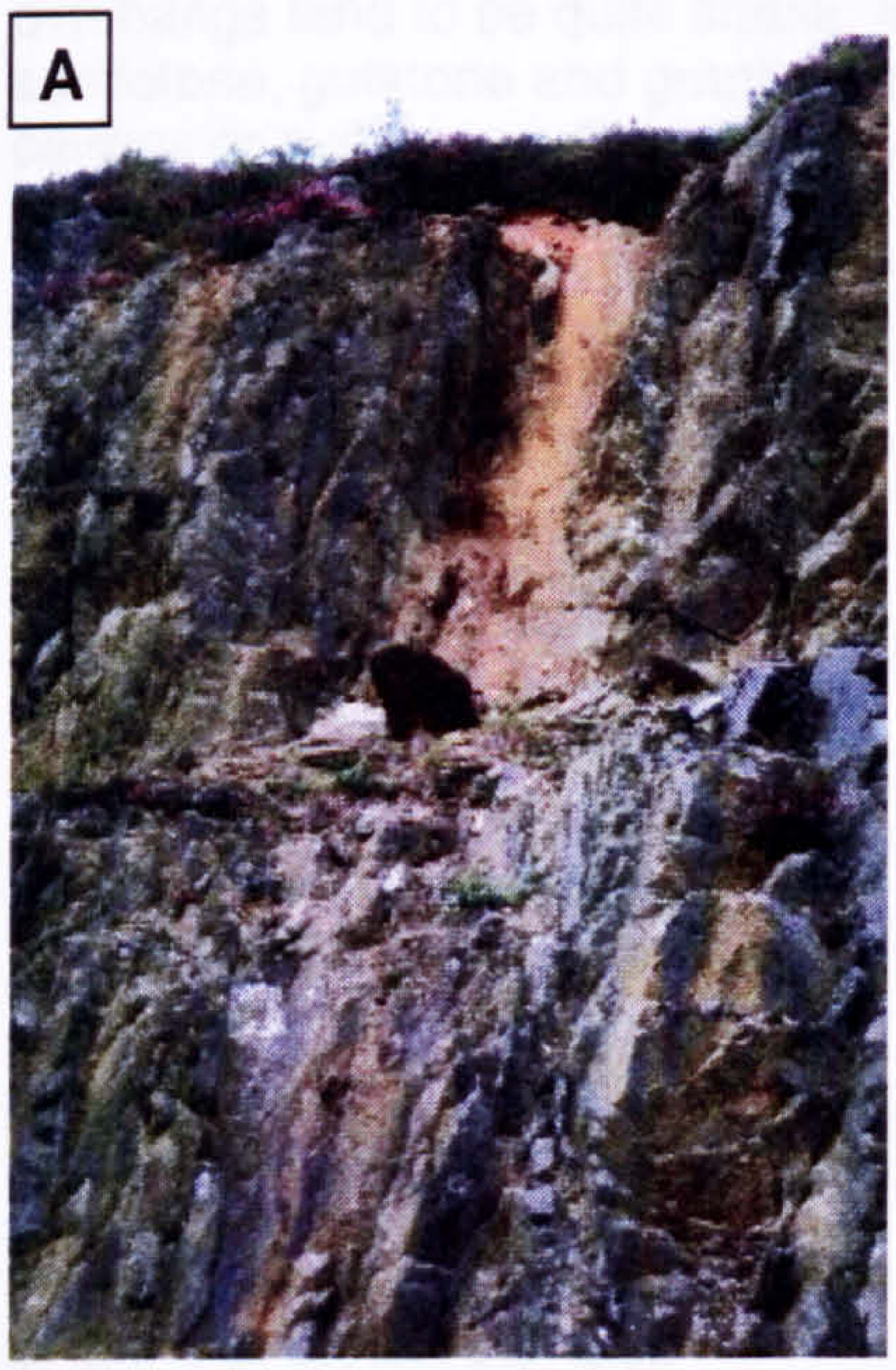
8.7.1 Diurnal fluctuations

Diurnal fluctuations can be regarded as those which occur (i) on a daily basis, (ii) at different times within a single day, (iii) on some days but not others. A variety of diurnal fluctuations can affect weathering and erosion processes.

EROSIONAL LANDFORMS: CHUTES

Chutes are quasi-channels down which material is transported. They might or might not be produced as a result of deterioration, but play an active role in the transport and re-distribution of material detached by deterioration.

A



Erosional chute (A)

Erosional chutes occur in weak, erodible rock, in which material is eroded by surface water runoff. Dissolution might also produce erosional chutes in soluble rocks. These chutes tend to be shallow and poorly defined. Wash erosion is the deterioration mode most often in evidence, but grainfall and grain ravelling might also occur when the material is dry. In severe cases, erosional chutes might provide conduits for small debris flows.

B

Fracture chute (B)

Fracture chutes occur in a wide variety of rocks in which fractures have been enlarged by wall breakdown, solution or other processes. Block wedging is commonly observed in fracture chutes. Growth of vegetation roots in fractures can also enhance both physical and chemical weathering of walls. Scaling of fracture walls commonly occurs, as well as grainfall and grain ravelling in granular rocks.

C

Structural chute (C)

Structural chutes occur in strong, competent rock masses, particularly where sub-horizontal layers are present due to folding. They are not formed by deterioration, but are a function of the rock mass structure and the form of the excavated slope profile. The velocity and nature of material transport down a structural chute depends on its gradient and width. Steep chutes encourage fall or rolling of rock fragments while shallower gradients encourage gradual creep or sliding of material. Wash erosion might occur in both cases. Shallower gradient structural chutes are characterised by an accumulation of debris and fines with organic matter where vegetation becomes established. In such cases the debris is stable and inhibits rapid movement of fresh debris along the chute.

Plate 8.1 (A to C) Deterioration morphology: Erosional landforms I

EROSIONAL LANDFORMS: OVERHANGS

Solution or weathering overhang

Solution overhangs occur rarely in soluble rock due to long term dissolution of material. Because the process relates to material weathering and not rock mass properties (eg fractures), these overhangs tend to be quite stable. In rocks susceptible to granular disintegration and spalling (eg sandstone, gritstone and granite), tafoni might form. These are large (typically >1m), stable cavities or overhangs. Deterioration associated with tafoni include grainfall, grain ravelling, scaling and flaking.

Erosional (composite) overhang (A, B)

Erosional overhangs occur in composite rock masses where weaker material is eroded, undermining more competent material above (**A**). They are the most common type of overhang. They present a real danger of collapse, sometimes producing large falls of rock. Isolated falls of stone or blocks can also occur. Erosional overhangs also occur very commonly at the top of slopes (**B**). Soil, reinforced by vegetation roots and organic matter, can be more competent than material forming the bedrock-soil boundary below it. Sporadic fall of clods of root-bound soil is common. The consequences could be serious if large scale undermining of a mature tree occurred, for example.

A

B

C

Structural overhang (C)

Structural overhangs occur in strong, competent rock masses, particularly those which are very thickly bedded or otherwise layered. Occasionally, structural overhangs form in the hinge areas of folded rock. They are a function of the rock mass structure and the form of the slope after excavation. Minor scaling or occasional stonefall from the underside of overhanging blocks can occur. Collapse is only likely if vertical cracks develop at the rear of the overhang, perhaps due to surcharge from deterioration above.

Plate 8.2 (A to C) Deterioration morphology: Erosional landforms II

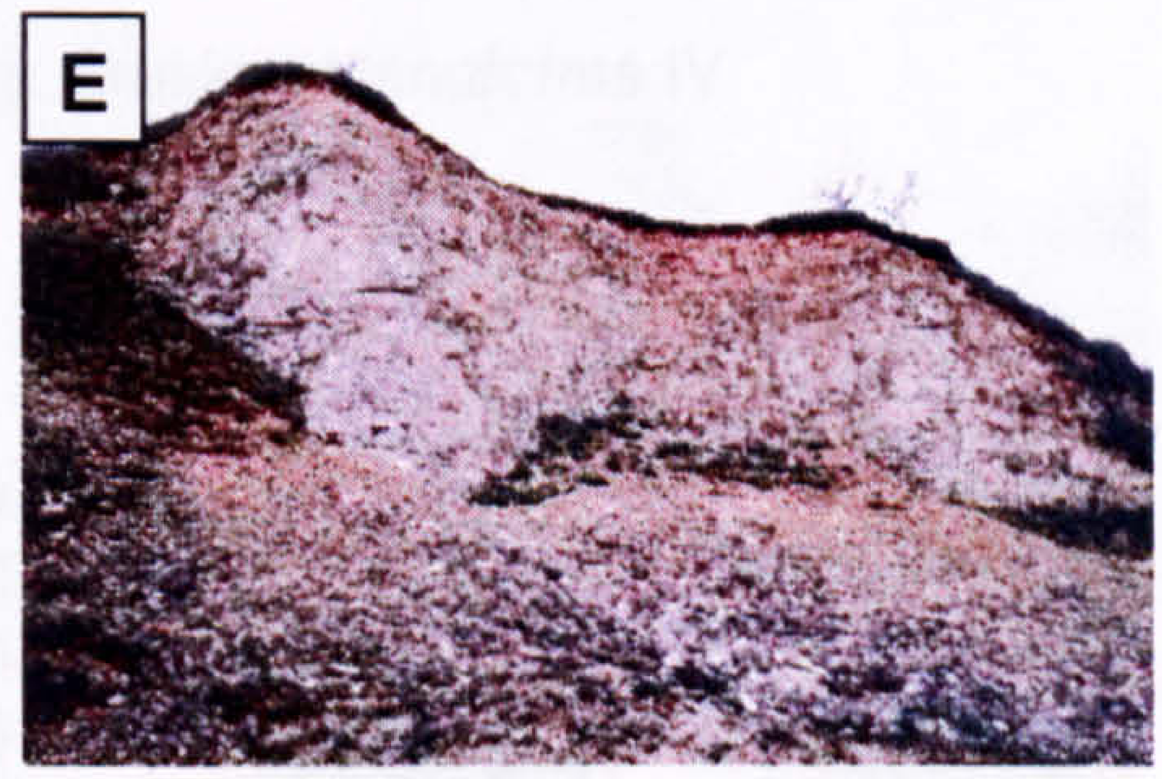
EROSIONAL LANDFORMS: CAVITIES

Honeycomb weathering structure (A)
Honeycomb weathering forms in rocks susceptible to granular breakdown (eg sandstone, gritstone and granite). Grainfall, grain ravelling and scaling occur and minor wash erosion might be present.

Solutional cavities (B)
Micro-solution pits and cavities can form in soluble rocks. These are also produced by bio-erosion.

Bedding plane cavities (C)
Small scale cavities often form along bedding planes in moderately weak rock. These result in very localised grainfall and grain ravelling.

MACRO DETERIORATION LANDFORMS

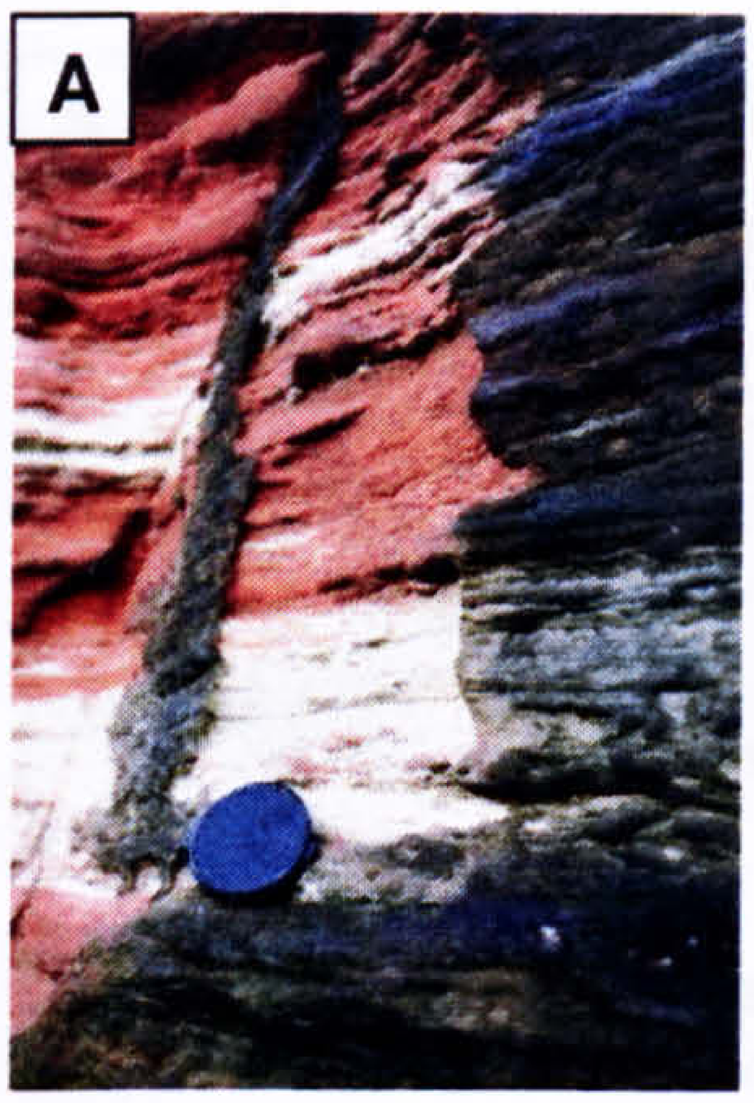


Palaeo-weathering features (D)
Palaeo-weathered profiles can be exposed upon excavation (eg deeply weathered rock, well developed karst forms, corestones and periglacial weathering forms).

Karstification (E, F)
In strong, soluble rocks such as crystalline limestone and hard chalk, large scale landforms develop due to deterioration, enhanced by blast shattering of the rock mass (Gagen 1988). Buttress and headwall sequences can be formed (**E**), influenced by the location of blast fracture cones (**F**) around drillholes. Collapse dolines might also be formed at the top of slopes. Progressive deterioration occurs by frequent rockfalls, and stone and block ravelling. Incipient forms of karst landform might be present in younger slopes and in weaker rock masses.

Plate 8.3 (A to F) Deterioration morphology: Erosional landforms III

EROSIONAL LANDFORMS: SURFACE SCARS



Surface scars occur in most rocks and represent the loss of material from the slope. The size, nature of staining and depth of scars indicate the nature of deterioration which has occurred. It might not be possible to determine if blocks were removed simultaneously, or by progressive ravelling, but reference to the spatial distribution of scars on the slope can help to discriminate.

Scars which cover a large area but which are extremely shallow indicate scaling (**A**). Where a little deeper, slabfall is suggested. Large, deep scars indicate loss of a substantial volume of material in a single event – ie rockfall (**B**) or debris flow, whereas small, deep scars indicate the fall or ravelling of individual stones or blocks (**C**). A large, planar scar along a discontinuity could indicate rockslide. For some deterioration modes, scars might not be evident. These include solution, flaking, and grain loss.

Plate 8.4 (A to C) Deterioration morphology: Erosional landforms IV

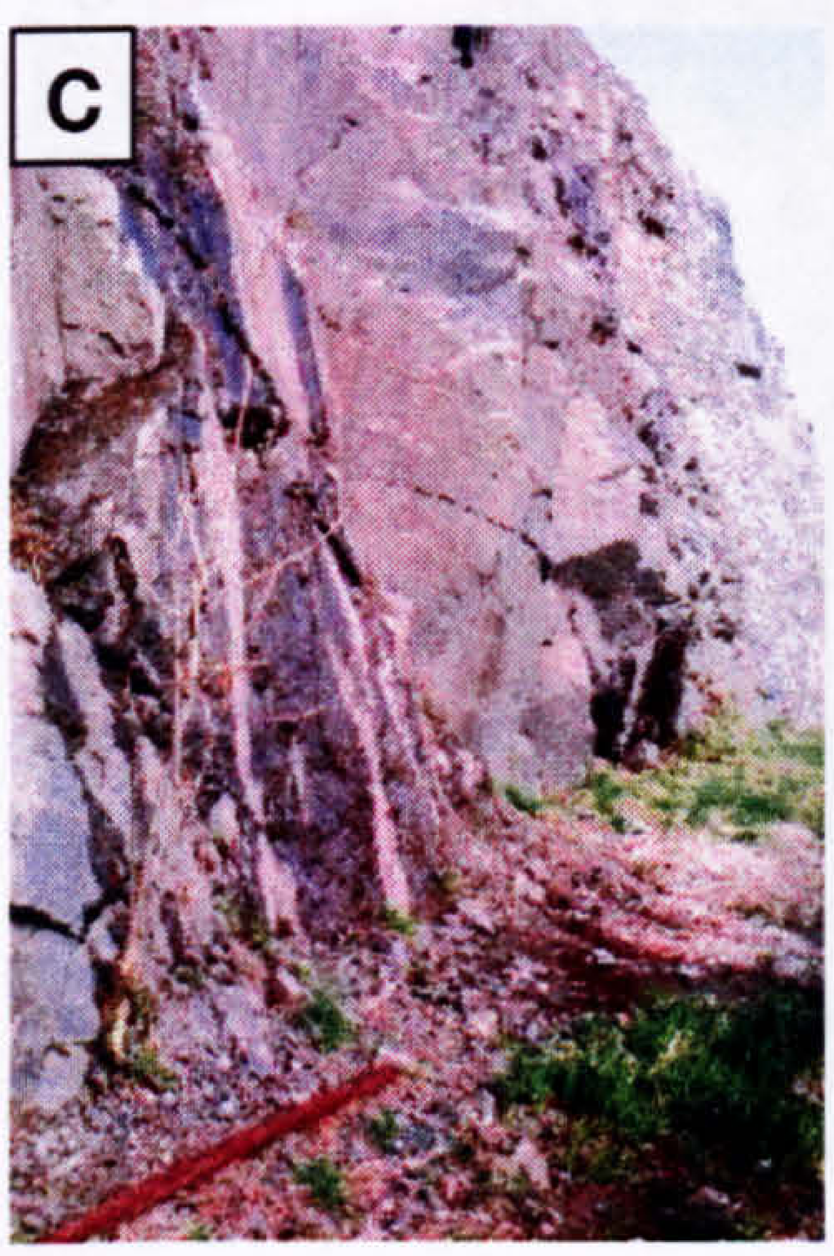
PROCESS INDICATORS: VEGETATION

Vegetation is found growing in a wide range of rock types. It is the presence of suitable growth medium and environmental conditions which control their ability to establish and flourish. cracks are exploited by the roots of mature trees (**A**) and woody roots are commonly found in association with intensely fractured areas (**B**).

Plate 8.5 (A and B) Deterioration morphology: Process indicators I

PROCESS INDICATORS: WATER FLOW


Evidence for surface water flow includes ripples and other flow structures in fines deposits (**A**); micro-solution pits and runnels (**B**); groundwater seepage (**C**); moss and algae growth on rock surfaces (**D**); stained and discoloured rock surfaces; penetrative discoloration; the presence of vegetation; flattened or 'draped' grass (**E**); deposits of fines washed from upslope (**F**); laminae, thin beds and fossils protruding due to water erosion (**G**); and channels or other pseudo-channels formed from solution, erosion, or in man-made channels (**H**). Water is essential for most physical and chemical weathering processes, and thus plays a role in most deterioration modes.



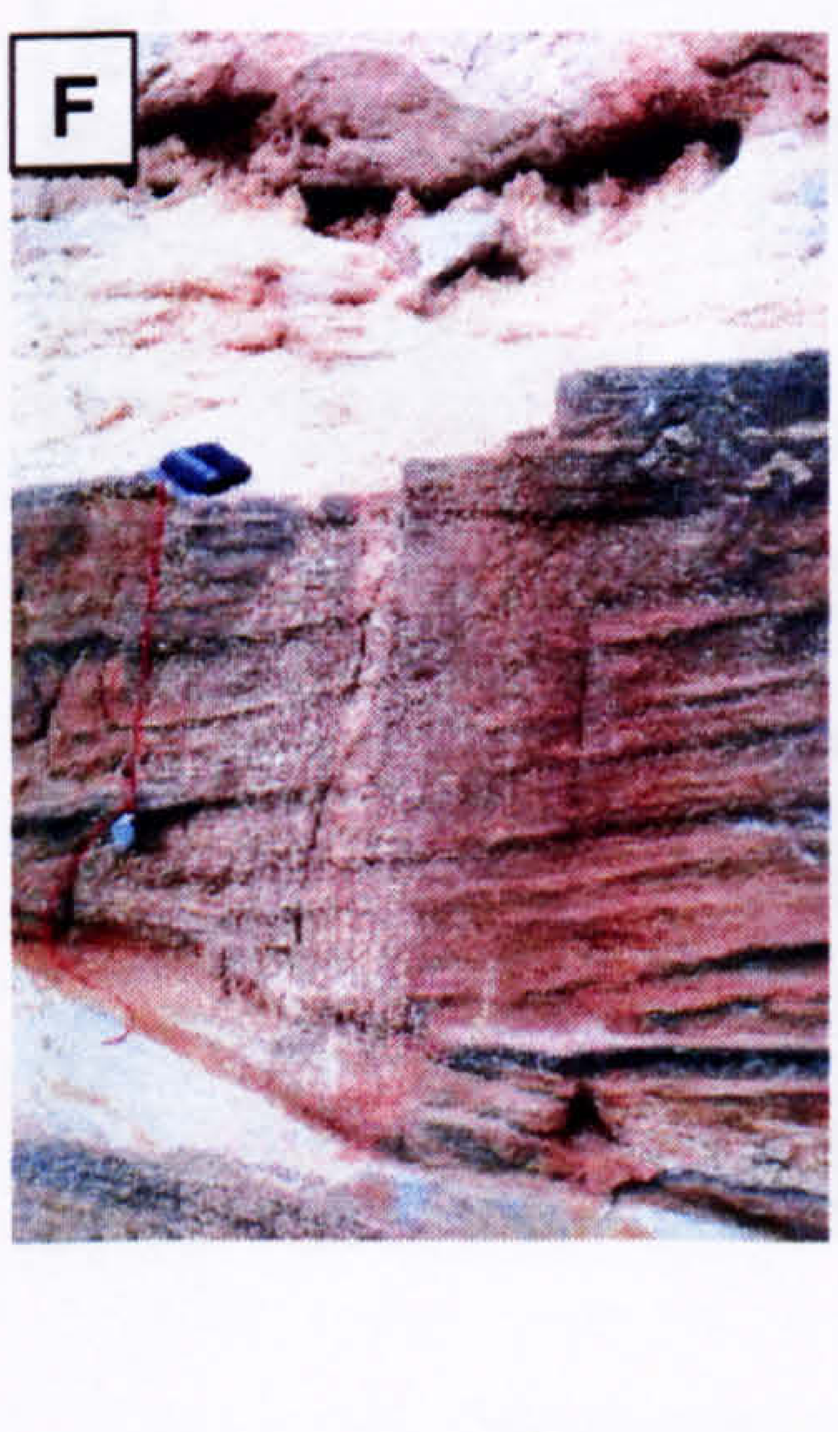
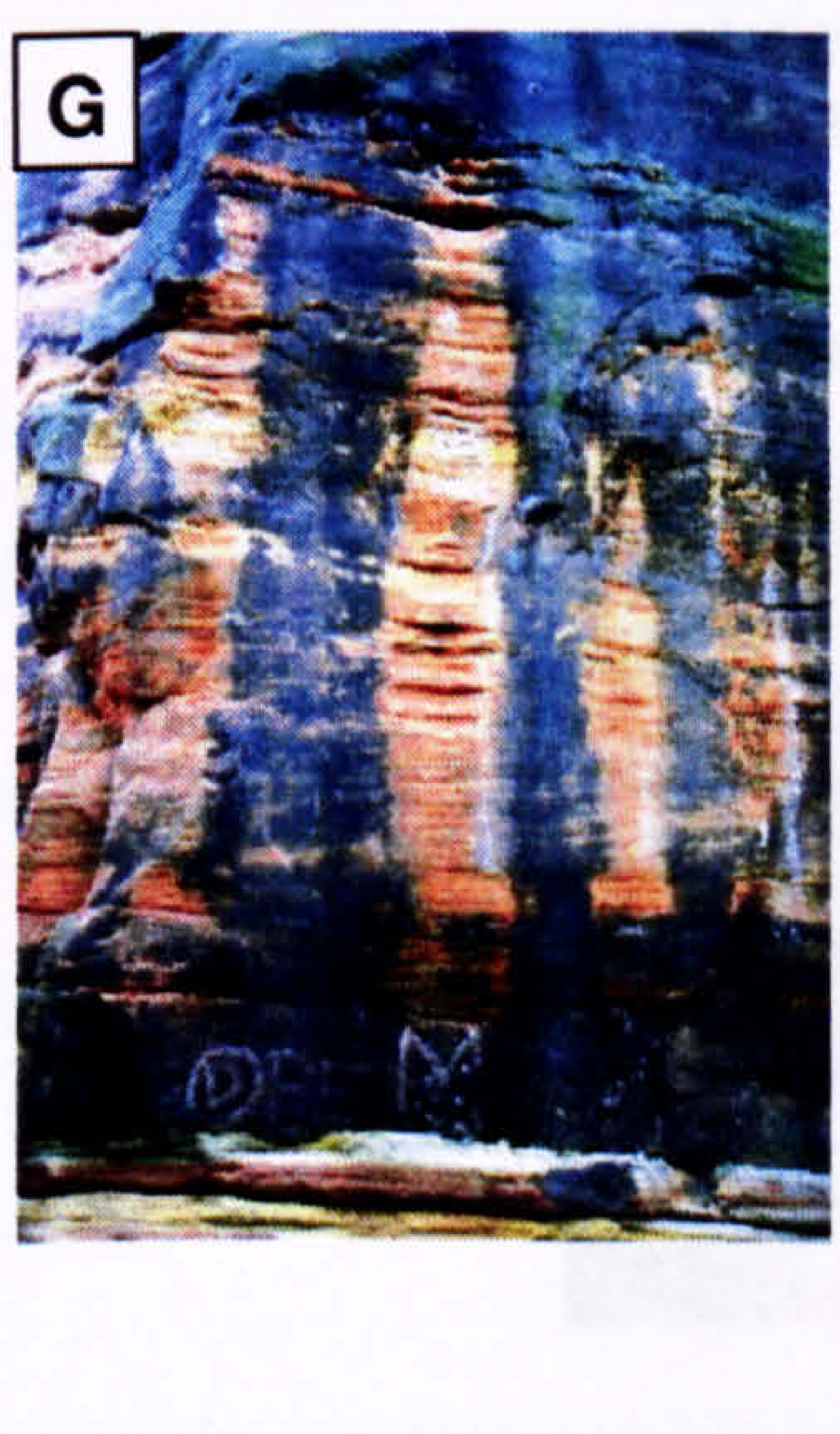
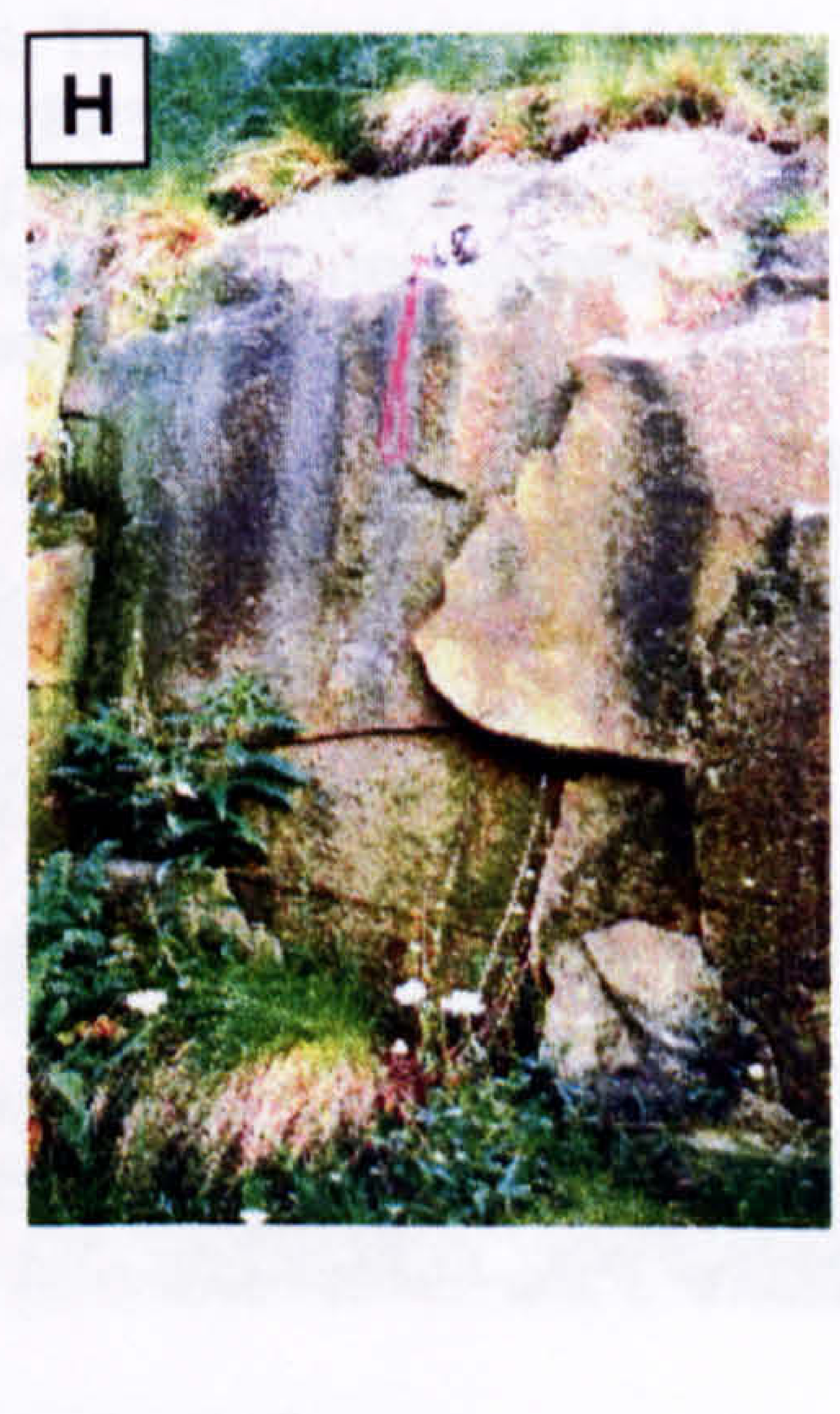




Plate 8.6 (A to H) Deterioration morphology: Process indicators II

PROCESS INDICATORS: IN SITU BREAKDOWN

IN SITU DECOMPOSITION

Evidence of in situ decomposition includes: incipient corestone development (**A**); dissolution (**B**); honeycomb weathering and cement decomposition. These processes tend to be associated with deterioration modes which focus on material properties (eg grainfall, grain ravelling, wash erosion, scaling and solution).

IN SITU DISINTEGRATION

Evidence of in situ disintegration includes: granular disintegration (**C**); stress release (rebound) fracturing (**D**); blast induced fracturing (**E**); spalling (**F**); dissolution of fracture walls; onion skin weathering; exfoliation; fragmentation associated with vegetation roots. These processes tend be associated with deterioration modes which focus on mass properties (eg stone and blockfall and ravelling, rockfall, slabfall, toppling and flexural toppling.

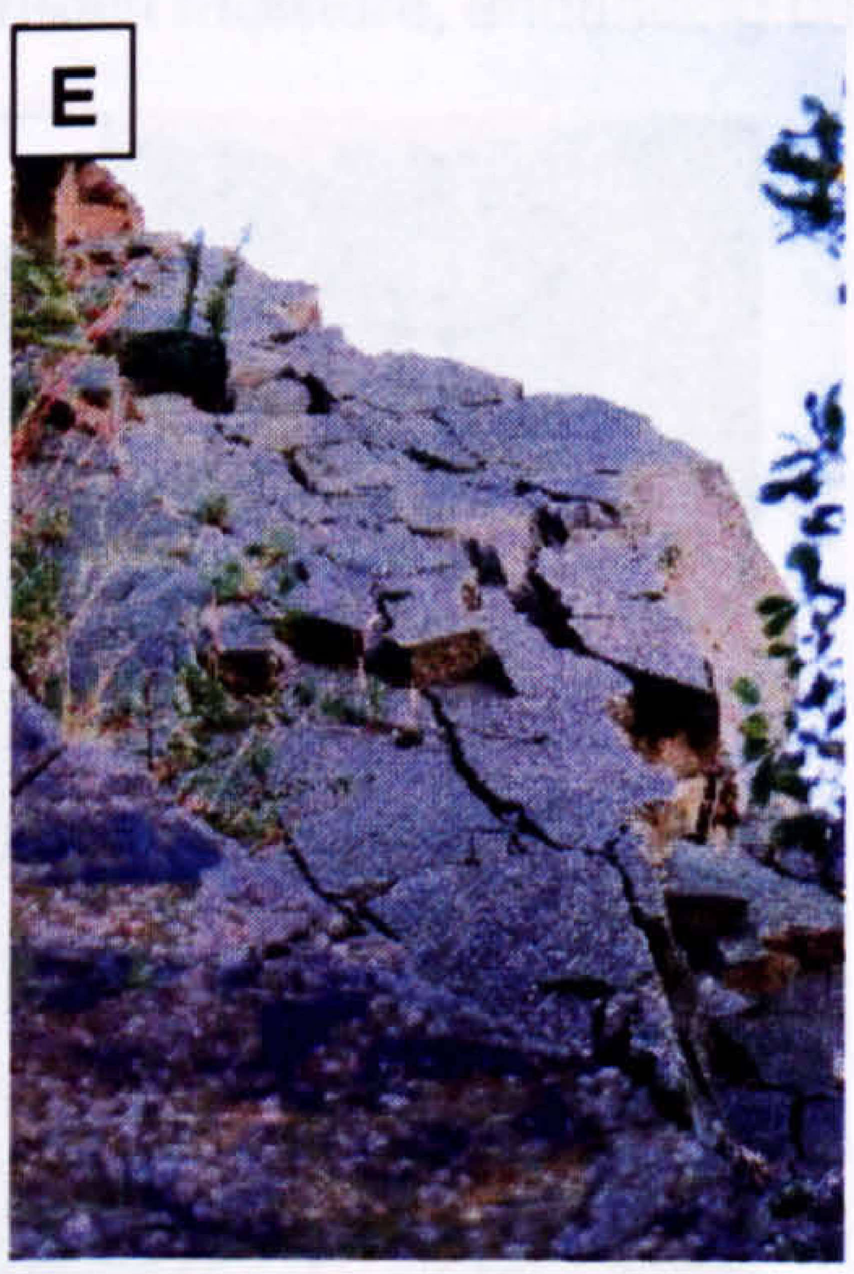


Plate 8.7 (A to F) Deterioration morphology: Process indicators III

DEPOSITIONAL LANDFORMS

DEBRIS PILES

A wide scatter of debris and extensive deposits of debris on slope ledges indicate semi-continuous fall by ravelling (**A**) and flaking (**B**), or wash erosion where the debris is fine (**C**). Platey material and individual grains tend to form steep, concentrated debris piles. Flatter debris piles with more spread usually indicate fall of material in a single event (eg rockfall (**D**), debris flow (**E**), and rockslide).

FRACTURE INFILLING

Fracture infilling occurs in open fractures where detached material moves downslope from above, but is particularly notable in widened fractures such as fracture chutes. Grainfall, grain ravelling, scaling of fracture walls and minor wash erosion are common, but occasional stone or blockfall also occur. The fracture infilling might have a cohesive effect, increasing stability, or retain moisture, enhancing chemical and physical weathering processes (**F**).

SCATTERED AND ISOLATED DEBRIS

A general scatter, or isolated stones and blocks at the foot of the slope, indicates stone or blockfall (**G**). Where the material is variable in size it might indicate a variety of controls on deterioration. Impact marks on intact rock or road pavements might be evident.

Plate 8.8 (A to G) Deterioration morphology: Depositional landforms

Meteorological conditions: These include variations in temperature, precipitation, humidity and solar radiation. Repeated temperature cycles about 0°C might be necessary for freeze-thaw weathering, while cyclic wetting and drying might be produced by precipitation and subsequent drying out by solar radiation. The minimum recurrence period for freeze-thaw cycles is usually 24 hours whereas the latter can occur several times within a single day. *Dynamic stresses:* Traffic flow, that is, type and frequency, changes through the day, both in terms of the day-night contrast, and also in terms of peak-time and non peak-time flows. Other dynamic stresses such as quarry blasting follow a similar pattern through weekdays, varying at the weekends. During a single day, blasting and its potential effect on slope deterioration is clearly intermittent. *Surface and groundwater runoff:* These are inextricably linked with meteorological conditions, though there is often a time lag between precipitation and groundwater seepage. *Direct disturbance:* Disturbance from human and animal activity is likely to be intermittent and extremely variable from day to day. Disturbance from active erosive processes such as basal undercutting will be semi-continuous, only varying diurnally in response to some of the factors already considered (eg precipitation and groundwater flow).

8.7.2 Seasonal and annual fluctuations

All of the factors considered in 8.7.1 also vary on a seasonal basis. Additionally, the greater number of freeze-thaw cycles in early spring and late autumn might explain why rockfalls are commonly more frequent at these times of year (Schumm and Chorley 1966; Douglas 1980). Although Luckman (1976) found that a secondary peak in rockfall activity correlated with summer storms. In cooler or high altitude climates, increased rockfall activity has been associated with periods of thawing (Rapp 1960; Matsuoka and Sakai 1999). The changing seasonal weather also has a direct impact upon vegetation growth, limiting the growing season from March to October for most plants. This means that weathering processes relating to plant growth are unlikely to be active during the winter months. Deciduous vegetation loses its leaves in winter through to spring and this might affect the shading and degree of exposure for a rockslope. Another factor potentially affecting rockslope deterioration is the potential for plants to die due to summer drought or particularly severe winter weather. The rootball of dead, woody plants can eventually collapse, taking with it some of the slope material. Direct disturbance from engineering works such as slope maintenance is likely to operate on a seasonal or annual basis. Processes such as solution and wash erosion might be active throughout the year, particularly after periods of wet weather.

8.7.3 Occasional or intermittent fluctuations

Some of the factors already mentioned influence rockslope deterioration on an occasional or intermittent basis. Lower frequency events such as a ten year rainstorm could cause significant erosion and undermining of slopes. Alternately, a particularly severe winter or drought-ridden summer could intensify some weathering processes. Several other scenarios are possible. For example, a shelterbelt could be felled or very severely pollarded, thus changing the exposure conditions of a slope in the short to medium term. Removal of woodland could also increase surface runoff and associated erosion. A new structure could be erected which changed the shading or exposure conditions of a slope. Direct engineering works such as implementation of stabilisation, protective measures and maintenance operations clearly have a direct impact on rock mass and material properties and slope geometry. This would also be true for a major engineering project such as slope widening or re-grading. A change in land use might also affect rockslope deterioration. For example, an abandoned quarry might be re-developed into a recreational facility in which direct face access was promoted (eg rock climbing, fossil collecting).

8.7.4 Long term fluctuations

These include the effects of long term landscape denudation, seismic activity, uplift and erosion, stress release and climatic change. The potential effect of climatic change should not be underestimated given the significant changes in meteorological conditions already apparent.

ROCKSLOPE DETERIORATION ASSESSMENT: STAGE THREE

8.8 Preventive and Remedial Treatment for Deteriorating Rockslopes

It is the general premise of RDA that the decision as to what mitigation and maintenance treatments to apply to a rockslope depends on a combined understanding of (i) the probability and severity of deterioration (ie risk) *and* (ii) the nature of deterioration (ie the hazard). The guidance given in RDA also presumes that the *consequences* presented by the hazard are unacceptably high. As stated earlier, though, this is a value judgement which must be made by those responsible. In recognition of the importance of both risk and hazard, selection of appropriate slope treatments in RDA is dealt with in two stages. In the first stage, the RDA_A Class is used to suggest a general approach to mitigation. In the second stage, a more detailed matrix of treatment measures is provided which relates RDA_A Class to deterioration mode. In this, specific treatment measures are suggested, pertaining not only to the likely risk of deterioration, but also to the likely characteristics of the deterioration mechanisms involved. Clearly, this guidance can be used to assist in the evaluation of potential capital and ongoing maintenance costs associated with a proposed rockslopes, and can therefore assist in the pre-excavation design and planning process.

Selection of treatment measures and design of a maintenance programme also depends on a variety of other factors including (i) cost; (ii) availability of materials; (iii) aesthetic or other environmental impact; (iv) availability of expertise; (v) conditions set out in planning permissions; (vi) local political issues; (vii) land use; (viii) land ownership and associated constraints; (ix) long term management and aftercare commitment; (x) safe access to the slope. Guidance given in RDA is independent of these considerations, which must be considered locally.

8.8.1 Rockslope stabilisation, protective works and maintenance operations

Deterioration of rockslopes can be mitigated using a variety of approaches. Slope treatment can be *reactive*, that is, works are carried out in response to infrequent or minor deterioration of a slope. Works can be *passive*, where the consequences of deterioration are reduced by containment and protection. A *semi-active* approach can be adopted, where the materials forming the slope are either improved or reinforced. A policy of *active intervention* can be adopted whereby substantial slope support, buttressing and retention are introduced. As a final alternate approach, if it is accepted that the consequences of deterioration are either too severe to be acceptable or to be mitigated successfully, then major *slope re-design* can be undertaken. A number of reviews of rockslope stabilisation and treatment measures have been published, including Fookes and Sweeney (1976); Peckover and Kerr (1977); Dubin et al (1986); Martin (1988); Fookes and Weltman (1989); Giani (1992); Dixon and Cox (1993) and Abramson et al (1996). A detailed review will not be repeated here, but some of the principle types of mitigation measure appropriate to each of the above approaches is briefly described, with information being based largely on the above-named review articles and field observations.

8.8.1.1 Reactive approach

In certain situations, it might be appropriate to literally 'do nothing' about deterioration. For such an approach to be viable the potential consequences arising from deterioration need to be acceptable. The essence of the reactive approach is that maintenance works and stabilisation measures are implemented on an 'as-needed' basis. Infrequent slope inspections can be undertaken to identify any maintenance requirements and to pre-empt problems. However, drainage measures are usually constructed immediately following excavation as a standard precaution, regardless of deterioration potential.

8.8.1.2 Passive approach

The essence of the passive approach is that deterioration is allowed to happen but its consequences are minimised by containment and protection. One of the simplest ways of achieving this is to increase the standoff distance between the slope and the potential casualty. An example would be to build a highway with a particularly wide verge. For some disused quarries where there is informal or formal public access, fencing and warning signs can be used to deter entry to danger areas. Protective barriers can also be used to prevent damage to structures by falling debris.

If deterioration is allowed to occur, then it is usual to adopt a more formal programme of maintenance works such as face scaling and clearance of debris. Loose or unstable material (and vegetation in some cases) lying on the surface of a slope can be removed using simple hand scaling tools and a hydraulic lift for access if needed. This includes debris accumulations on ledges and potentially dangerous overhangs. One difficulty encountered in scaling highly fractured rockslopes is that there is often no distinct boundary between stable and unstable materials. There is then the risk that stable material will be disturbed unintentionally (Dubin et al 1986). In order to plan scaling and associated works, a plan of regular inspections can be followed, perhaps including some limited monitoring of critical blocks, either visually or by using instrumentation. The process of scaling might lead to critical blocks or areas being identified which necessitate mechanical excavation, perhaps using a pneumatic hammer or drill (Dubin et al 1986).

The most common solution for deteriorating rockslopes is to absorb the energy of falling material by constructing a rocktrap ditch, often with a protective fence or other protective barrier. Design data and charts have been produced which offer guidelines on the relationship between the depth and width of rocktrap ditches and catch fences in relation to the height and angle of slope (Ritchie 1963; Fookes and Sweeney 1976; Mak and Blomfield 1986; Whiteside 1986; Fookes and Weltman 1989). Sophisticated rockfall trajectory software programmes are now available to assist in slope design (eg Robotham et al 1995). Rockfall shelters can also be constructed to protect people, vehicles or structures at the foot of the slope from falling debris (Giani 1992). Another solution is to hang galvanised wire mesh netting over the slope face. If weighted at the bottom, material is thus prevented from being thrown out from the face and allowed to collect in a purpose made ditch at the foot. Some modern wire mesh nets are designed to catch blocks with energies up to 3500kJ. These not only reduce the velocity of falling debris but limit its trajectory. Netting is particularly useful where fracture spacing is very close, such that individual blocks cannot be retained or reinforced using other methods (Dixon and Cox 1993). Netting is usually fixed to the face with dowels or short anchors.

A further, environmentally-friendly method of absorbing the impact and energy of falling debris is to establish a dense cover of low growing shrubs and herbaceous plants at the foot of the slope. Trees and shrubs which reach a significant height could intercept falling material in such a way that it is thrown out from the slope. This is likely to be particularly hazardous. Vegetation used in this way might not be appropriate for all localities, and is certainly not recommended where a substantial accumulation of debris is anticipated (eg flaking, ravelling, rockfall). But for less severe deterioration modes such as stonefall, wash erosion, scaling and grain ravelling it might be useful.

8.8.1.3 Semi-active approach

The aim of the semi-active approach is to reinforce the slope, either by improving the strength of the material, or by improving continuity of the rock mass. The latter can be achieved by reinforcement of individual blocks using dowels or rock anchors. *Dowels* are made of steel and are in the form of bars or rods, drilled or grouted into the surface at close spacing, and near vertical to maximise shearing resistance for sliding blocks. Their strength enables them to resist bending moments, and tensile or shear forces. Dowels are not tensioned and are therefore passive, in other words, they do not actually do any work until the slope moves and then the tensile strength of the dowel is mobilised. Dowels can be used in both homogeneous and heterogeneous materials. They are often used in conjunction with shotcrete (see below). Dowels are cheap to use, requiring only light construction equipment. They can therefore be adopted on small slopes with difficult access. *Rockbolts* are also steel rods inserted into pre-drilled holes usually 3-10m in length. They are often pre-tensioned and can be fitted with instrumentation for monitoring purposes (Dixon and Cox 1993). They can be applied as a general grid over the face, known as pattern bolting, (more applicable for general instability) or applied to specific critical blocks. Since they are tensioned, bolts are active in preventing block movement. Rockbolts are vulnerable to corrosion if not properly treated. This can lead to enhanced material breakdown along the rod, and behind the head of the bolt at the surface. In severe cases, steel cables 10-40m in length can be used, but these would be mainly applicable to large scale or deep-seated instability. A further method of reinforcement is the *grouting or sealing* of fissures and voids. This will also prevent water access to critical fractures and stiffen surrounding rock. This can be achieved with shotcrete, clay, concrete or bitumen. In particularly large fractures, bulk material can be used to fill the void prior to sealing.

Temporary reinforcement can be gained by cable or chain lashing of individual, large blocks. The cable or chain can be anchored, or fixed to intact rock with dowels. Anchors are generally avoided, particularly for small scale deterioration problems, because of the potential for corrosion and long term maintenance and monitoring requirements (Dubin et al 1986).

The strength of the material can be improved by applying a surface protective cover. The materials used in any such treatment must be sufficiently durable to resist prevailing weathering conditions. The selection of suitable materials is influenced by the area to be covered and whether or not aesthetic considerations are important. Several techniques are possible: *Shotcrete* is conventional concrete, applied by pump action to the slope surface. It is often reinforced with steel mesh or fibres (eg Dixon and Cox 1993) and weepholes are installed to assist drainage. Shotcrete serves two functions, it binds loose material together and offers protection against the influence of weathering. In particular, it reduces infiltration of surface water. Shotcrete is often used in combination with rockbolting, with the shotcrete being sprayed over the boltheads after installation. One of the attractions of using shotcrete is that large areas can be covered quickly and cheaply. Its appearance can be improved with the use of a colouring agent. *Masonry blocks* are ideal where aesthetic considerations are important, but they are more expensive than other forms of surface protection. Stone blocks are bedded on gravel, sand or other free-draining material, and side joints are mortared. Again, weepholes would normally be installed at the top and base of the slope.

In suitable materials, *vegetation cover* can also be established in conjunction with geotextiles (eg coir netting or geosynthetic materials). This also applies to soil-like overburden materials at the top of a rockslope, which might nevertheless be degrading, contributing to the general deterioration hazard. Dubin et al (1986) also report the successful use of a proprietary spray-on seeded binder on near-vertical, moderately weathered rockslopes in Hong Kong. The binder is resin-based, with organic fibrous matter (as a cultivation base), plant seeds and a cement binding agent. The material is sprayed onto the rock surface using a wet process similar to that used for hydroseeding.

Local rock mass continuity and improvement in material strength can be achieved with use of *dentition,* particularly for individual blocks, small overhangs and weak zones or layers. The area is trimmed back and cleaned up and packed with a filler or concrete. Usually, the surface is faced with masonry or reinforced concrete and weepholes installed. Dentition is often used in conjunction with dowels (Fookes and Sweeney 1976). For very small areas, mortar can also be applied directly to the rock.

8.8.1.4 Active intervention

The aim of active intervention is to provide support to weak parts of the slope using buttressing, retention, application of dentition (covered in 8.8.1.3) to large areas, and underpinning. *Underpinning* is appropriate for larger cavities and overhangs using concrete or timber beams, or steel girders in some cases. These can be anchored in place or fixed to stable rock with dowels (Fookes and Weltman 1989). *Buttressing* is often used to support zones of highly fractured or weathered material such as are found in shear zones (Dubin et al 1986; Dixon and Cox 1993) and as such is ideal for the treatment of intensely fractured zones and erosion or fracture chutes. Buttresses are usually designed as gravity retaining structures (Dubin et al 1986) and usually contour the rock face. They can be constructed entirely of concrete, entirely of masonry, or a combination of both. They are fixed to the rock surface using dowels or bolts and can be reinforced. As with many of the treatment measures described here, drainage is essential and might be provided by a series of regular weepholes or inclined drains (Dixon and Cox 1993). Particular care is needed to ensure good drainage at the junction between buttressing and intact rock. If not, seepage erosion and enhanced weathering might occur, with the boundary zone acting as an erosion chute. An example of this from Jeffrey's Mount on the M6 can be seen in Plate 7.9.

Retaining walls are usually constructed as gravity structures (their resisting force comes from their dead weight), though tieback walls, anchored into the slope behind, are also used. In this case, their resisting strength comes from their anchorage into the slope beyond the deteriorating rock mass. This anchorage transfers the load via steel cables, rods or wires, which are grouted into strong bearing rock (Abramson et al 1996). Retaining walls would be most useful for supporting weakened slopes which are vulnerable to high magnitude events such as rockfall, debris flow, rockslide and flexural toppling. Any retaining structure must therefore be capable of withstanding overturning and sliding forces, as well as internal shear and deformation, and the ground on which it is located must have sufficient bearing capacity. Again, drainage behind and beneath the retaining wall is usually essential. Retaining walls can be constructed using concrete (pre-cast and cast on site), gabion baskets, and pre-cast concrete or timber crib blocks.

Crib walls are usually constructed with pre-cast concrete or timber beams and backfilled with granular fill. For shallower angle, planar slopes, the use of *grascrete* can also be effective (Dubin et al 1986). Topsoil forms the uppermost layer of fill to enable vegetation to establish where desirable. Although more commonly used to retain soil masses, *gabion units* can also be used for retention of small areas of weak and weathered material. These are rock-filled baskets which have the advantage of being strong, heavy, permeable and deformable (Hoek and Bray 1981).

8.8.1.5 Slope re-design

If rockslope deterioration is deemed too severe to mitigate in any of the above ways, or where mitigation works are not deemed to be reducing the consequences adequately, slope re-design might be the only solution to the problem. In the case of proposed slopes, where severe deterioration is anticipated, it might be possible to avoid the issue by re-location or complete re-design. However, this solution is rarely an option, except at the stage immediately following preliminary site investigation. Any later than this could incur significant wastage of resources. This approach is also unlikely to be viable in the context of mineral extraction since the primary choice of site will depend upon the quality and extent of the mineral resource available. A further option to minimise deterioration potential for proposed rockslopes is the selection of a less damaging excavation method (eg pre-split blasting). Again, this method would be inappropriate for use in mineral extraction where the principal objective is to maximise fragmentation. In theory, it could be viable for production of final 'restoration' slopes in worked out quarries, but aesthetic and cost considerations might prohibit this.

For existing slopes, the slope geometry may be modified. The gradient can be reduced either by re-excavation or addition of fill material, or benching can be introduced, where a slope is divided into sub-slopes of smaller dimensions. One drawback of modifying slope geometry in these ways, as well as the costs involved, is the need for additional landtake. Apart from the obvious legal, planning and economic implications of this, there are geometric factors to consider too. For example, the effect of introducing benching into a slope situated beneath rising topography will be to increase the total slope height. Also, unless bench surfaces incorporate rockfall protection (eg ditch or protective fence), they might cause falling debris to be thrown out further from the slope foot than would have otherwise have been the case.

The deterioration situation can be significantly improved by the removal of surcharge load at the crest of the slope. This is dependent upon access to the upper part of the slope with heavy machinery. Trees often add surcharge to a slope. The amount of load involved depends upon the mass of the tree, which includes the stem, the crown spread above ground and the root spread below ground. Normal loads of 0 to 2 kN/m^2 are not uncommon for forested slopes, but can be *much* higher for individual mature trees with a narrow root spread.

8.8.1.6 Slope drainage

Drainage measures have been ignored till now because they deserve special attention and because their use in reducing deterioration potential can be applied for any of the mitigation approaches described above. Slope drainage has several beneficial effects, it increases shear strength of the rock by reducing pore water pressure; reduces the unit weight of overlying material; reduces or prevents infiltration of surface runoff and thereby contributing to groundwater flow; and removes water from the surface and sub-surface, important contributors to many weathering processes. Drainage measures can be broadly classified in terms of surface and sub-surface methods. A schematic illustration of various drainage measures suitable for rockslopes is shown in Figure 8.29 (after Fookes and Sweeney 1976).

(a) Surface drainage

Surface drainage can be achieved by the use of shallow ditches or purpose made concrete or plastic channels. There are three main types. *Shallow collector drains* are situated at the slope crest or on the surface of individual berms. They are shallow, gravel or stone-filled ditches, usually 0.5-1.0m deep and lined with a geotextile fabric. Their primary purpose is to reduce infiltration of surface water into the slope. *Collector drains* are located on the slope surface. In soils, highly weathered rock and soil-like materials they can be constructed as before but set out in a herringbone pattern. In stronger materials they might simply take the form of vertical or sub-vertical pipes, often built into the slope and covered with masonry or concrete. Their primary function is to transport surface water or water collected up from crest drains, to diversion drains at the slope foot. *Diversion drains* are often constructed as pre-cast concrete channels, and are situated at the slope foot. Their main function is to collect water from the slope and transport it into a soakaway or into the mains system.

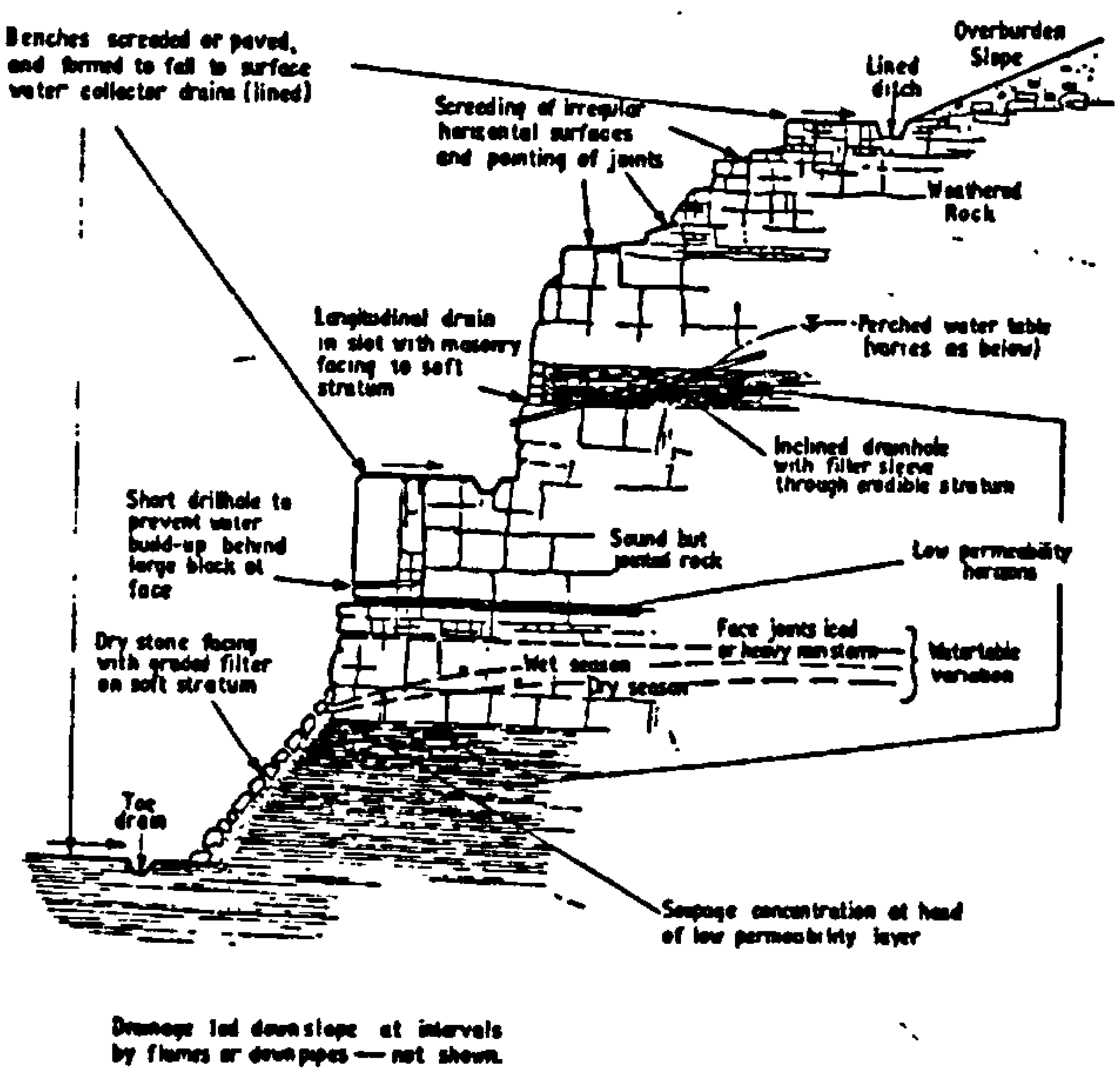


Figure 8.29 Schematic illustration of drainage measures suitable for rockslopes (*after Fookes and Sweeney 1976*)

(b) Sub-surface drainage

Sub-surface drains usually require excavation of the rock mass, either at the top of the slope, on its surface or at the foot. This is expensive and can lead to significant disturbance, so is only used in more severe cases and where there is an excellent chance of successful mitigation. Several types of sub-surface drain are possible but most are either applicable to soil slopes (eg trench drains) or to large landslide masses. The main options for rockslopes are the use of lined *cut-off drains* at or near the top of a slope to intercept groundwater flow where it occurs near the surface. They are usually situated parallel to the slope. *Inclined drains,* usually simple drillholes with a filter sleeve, are also commonly used in association with buttress walls or to drain weak strata within otherwise competent rock. Short drillholes or *weepholes* are usually used in conjunction with dentition and masonry walls.

8.8.1.7 Use of vegetation

The beneficial and adverse effects of vegetation on rockslope deterioration have been described in Chapter Six and will not be repeated here. However, mention must be made of the specific use of vegetation as a stabilisation tool in civil engineering, a discipline known as *bioengineering.* Vegetation has long been used in erosion control and its application in slope stabilisation is increasingly becoming apparent (Thomson 1988). Many bioengineering applications apply to soils, soil-like materials and highly weathered rock. However, vegetation can also be purposely established on rockslopes, either on fragmented screes or in specially created niches. These methods have been used in the restoration of worked out quarries to serve the dual functions of stabilisation and habitat creation. Attempts have also been made to utilise vegetation in rockslope protection for highways located in sensitive areas (Blunt and Dorken 1994). However, successful establishment of vegetation is governed by a wide range of factors pertaining to the tolerance and needs of the plants used; the nature of the substrate and its nutrient capacity; and the prevailing climatic conditions. Blunt and Dorken (1994) achieved limited success in their widespread seeding of shaley highway slopes and suggest that the primary reason for this was lack of suitable soil material. The site location in exposed, upland Wales might also have been a contributory factor. They recommend that pocket planting of trees and shrubs in specially prepared niches could prove to be more successful.

Some advantages of using vegetation for slope stabilisation are that it is usually cheap and easily available; it can be visually attractive; and it is relatively easy to put in place. Disadvantages are that vegetation can take a very long time to establish (though rapid growing species can be utilised); it can be damaged due to drought, blight, waterlogging, exposure and pollution; it can be difficult to establish in poor ground conditions, poor quality soils and on steep slopes. It is important to bear in mind that because it is a living material, a long-term programme of vegetation management and maintenance will be necessary.

8.8.2 Special considerations for quarry slopes

Assessment of the consequences of deterioration of *disused faces contained within working quarries* addresses similar issues to those for highway slopes, considered in section 8.8 above. The primary issues are safety of workers, both at the foot and above the face, and maintenance, especially where the fallout zone of a slope impinges on haul roads or other working areas. However, there is often much greater flexibility in a quarry environment to deal with problems arising from deterioration. For example, it is relatively easy to move an endangered haul road or working area to another part of the quarry. It is also fairly straightforward to prevent access to unsafe areas by vehicles, machinery and workers, as well as preventing public access. Operation of standard safety procedures in quarries might also help to limit the potential consequences of slope deterioration, for example, by restricting lone working, ensuring good communications between workers, providing first aid and emergency procedures and undertaking regular face inspections.

In *disused quarries*, the consequences of deterioration depend largely on the use to which to the land is put. However, planning conditions, local politics and environmental considerations might take on a much more influential role in terms of how deterioration is mitigated. The consequences of deterioration in disused quarries relate largely to after use. Disused quarries can be utilised for a variety of purposes, some actively encouraging public access and some permitting access on a more casual basis (Coppin 1981). What is more critical is whether or not the quarry face plays an intrinsic part in the activity. This is likely to be the case with rock climbing, SCUBA diving in flooded quarries, and geological conservation sites. The public are also likely to come into close contact with the face at a variety of other disused quarry sites such as industrial estates; forestry, caravan or car parks; and recreational sites where the face is incidental (eg country parks).

Mineral planning authorities are likely to have made it a condition of planning permission that once worked out, the quarry is restored in a particular way. Stabilisation measures utilised for road cuttings might not be appropriate because of the potential for aesthetic impact, cost, the lack of expertise available in the quarry industry, and the possibility of such measures interfering with intended after uses. For instance, measures which cover large parts of the slope surface (eg shotcrete) would clearly not be compatible with geological conservation. Indeed, slope deterioration can actually be a benefit for certain after uses. As already mentioned, weathering might enhance the appearance of a slope and this could be important for quarries located in sensitive areas. Alternately, a loose, weathered material presents a better medium for plant establishment and for nesting sites in quarries given over for wildlife conservation.

Certain 'treatment' techniques can also be adopted in disused quarries which would be unusual for road cuttings. For example, it might be possible to substantially re-grade slopes or undertake partial or full backfilling. Restoration blasting techniques (section 8.8.2.1) could be used to create a completely new profile, and the establishment of vegetation is much more likely to be a requirement rather than an option. Given these differences and the wider variety of considerations involved, these treatment measures are not specifically addressed in RDA, since their selection is less related to the risk or hazard of deterioration, and more to other factors. Some of the treatments unique to disused quarries are considered briefly below. This of course, does not negate the use of RDA for quarry slopes since it still enables evaluation of the probability and severity of deterioration, as well as the nature of the hazard. These factors will, despite consideration of a wider range of other factors, play an important role in determining the suitability of after uses, slope treatment and long term site management.

8.8.2.1 Restoration blasting

The *restoration blasting* technique was developed by Gagen (1986, 1988) and Gagen and Gunn (1987a, 1987b, 1987c, 1988) for use in limestone quarries and has subsequently been applied to other rock types. The original concept was to replicate natural 'daleside' landforms modelled on the White Peak of Derbyshire. This was to be achieved by application of specially developed blast specifications enabling the re-creation of daleside landforms with rock screes, buttresses and headwalls This led to research into the most effective means of vegetation establishment appropriate for the same environment. Trials included hydroseeding, pit planting of tree transplants, general planting of tree seedlings and hand sowing of tree seed (Gagen et al 1993). The idea was for restoration blasting to be carried out at the cessation of quarrying, prior to closure. A number of trials, with varying success, have been reported in the publications cited. Vegetation establishment has not always been successful and concerns have been raised over the stability of buttresses which are exposed on three sides (A. Kirk: Discussion in Wakefield et al 1992) and of the upper parts of headwalls and buttresses (Walton 1993a), particularly in the context of rockfall activity. A further concern of industry is the inevitable need for landtake. This means either, acquisition of, and planning approval for, extra land, or the surrender of recoverable mineral reserves (Wakefield et al 1992). Nevertheless, there has been significant interest in the technique and it has been applied with reasonable success in a modified form in other rock types and geographic locations.

8.8.2.2 Backfilling and re-grading

With increasing pressure on land, dual uses of land have been sought. One result of this is the increasing number of worked out quarries which have been re-developed for landfill. Not all rock types have been suitable for this in the past, but increasing confidence in the long term durability of geosynthetic and natural liner materials has enabled more and more quarries to be landfilled. It is not uncommon for both extraction and landfilling to operate simultaneously.

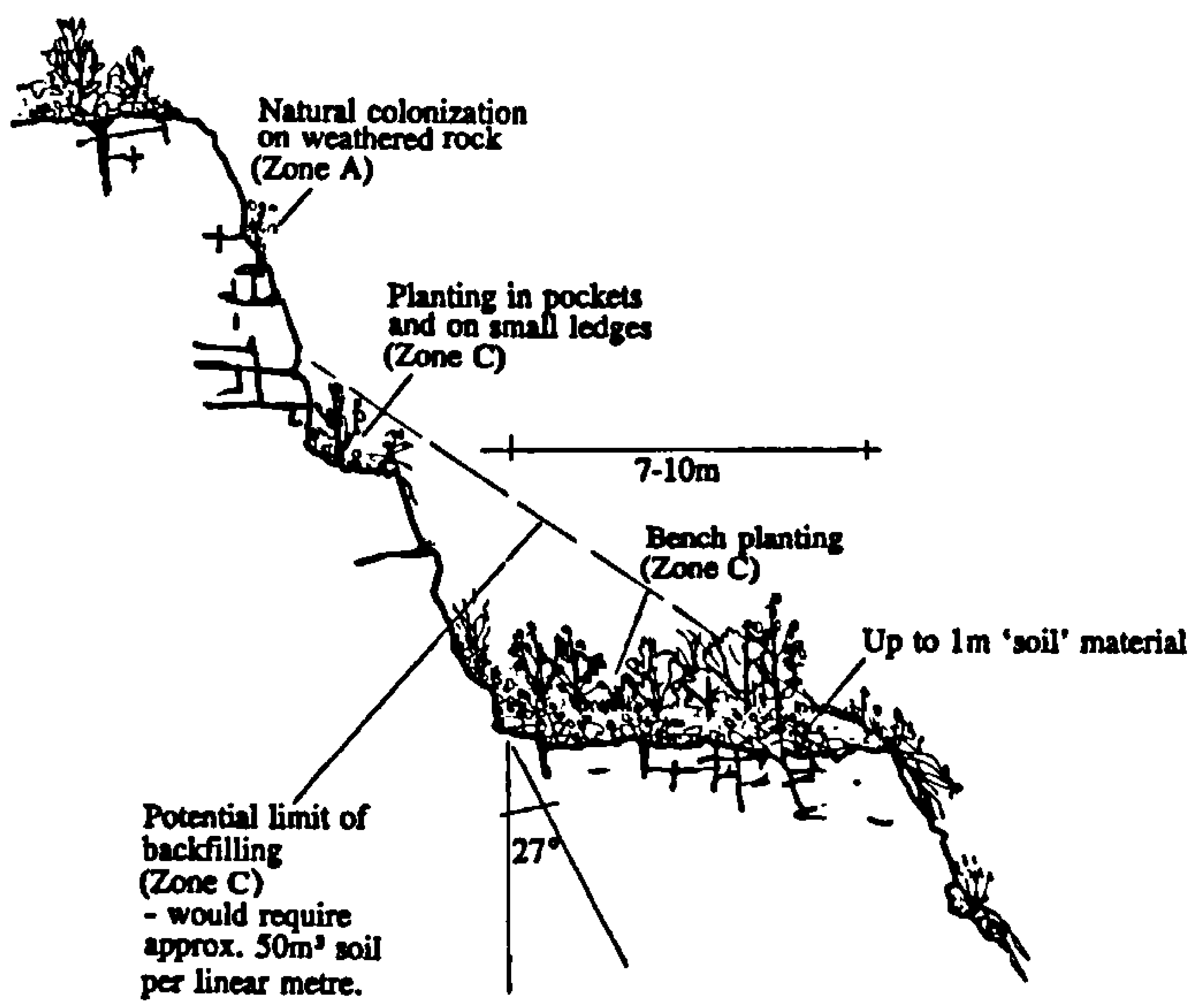


Figure 8.30 Pocket planting proposals for restoration of a limestone quarry

8.8.2.3 Vegetation establishment

Given the considerations in 8.8.2, it is actually desirable in many cases for vegetation to be established on disused quarry faces, despite the likelihood of associated deterioration. A range of techniques are available. Figure 8.30 is taken from a limestone quarry restoration scheme (details confidential) designed by the author in 1994 in which vegetation establishment trials were proposed and accepted by the Mineral Planning Authority. These largely involved pocket planting in specially created niches supplied with suitable cultivation material. Some quarry operators have experimented with extensive soil application and planting along quarry benches with no modification of the slope form (eg Whatley Quarry, Frome, Somerset). Others have used a modified form of restoration blasting. Instead of formally designing restoration blasts, rock piles left at the cessation of quarrying have been covered with subsoil and topsoil medium, and then seeded or planted (eg Minera Quarry, Wrexham, North Wales). At Llynclys Quarry near Oswestry in North Wales, substantial backfilling of quarried slopes has been undertaken. The surface of the backfill has then been thinly covered with fines taken from drained tailings ponds and left to colonise naturally. Because there is an excellent natural seed pool in the local area, natural colonisation has been both successful and rapid.

8.8.3 Application of RDA Class: general meaning of classes

Table 8.2 below indicates the general level of slope treatment required for each RDA_A Class. However, since treatment measures differ considerably for different deterioration modes, this table should only be used as a general guide. It could be used, for example, at the desk study stage of a proposed excavation in which several sites in similar rock masses were being compared. It could also be used to compare likely mitigation requirements of different zones within a single excavation. A more detailed indication of appropriate treatments, based on an understanding of the likely nature of deterioration, is given in Table 8.3, section 8.8.4.

RDA$_A$ Class	Adjusted rating	Level of risk	Approaches to remedial treatment*
1	<21	Very low risk	**Reactive approach:** Maintain or remedy as necessary. Examples include infrequent inspection and debris clearance.
2	21-40	Low risk	**Passive approach:** Control the consequences of deterioration by containment and protection. Examples include scaling; wire netting; rock catch ditch and protective fencing.
3	41-60	Moderate risk	**Semi-active approach:** Reinforce the slope and slope materials to resist processes of deterioration. Examples include surface protection (eg shotcrete, geotextiles and vegetation); dowels, cables, anchors and rockbolts; dentition.
4	61-80	High risk	**Active intervention:** Retain and support the slope. Examples include crib walls, gabions and buttresses; underpinning.
5	>80	Very high risk	**Slope re-design:** Examples include reducing slope gradient; benching; increasing stand-off; rockfall shelters.
* Approaches to remedial treatment are cumulative, ie a class 3 slope will require an 'active' approach *in addition* to measures indicated for classes 1 and 2.			

Table 8.2 General approaches to treatment of deteriorating rockslopes, based on stage one of RDA

8.8.4 Detailed treatment measures matrix

The matrix given in Table 8.3 is intended to act as a guide for treatment measures suitable for different deterioration modes likely to occur in rockslopes with varying levels of risk. However, it is important that the actual selection of works is based on detailed site appraisal and engineering judgement. It should be noted that certain classes of RDA$_A$ are unlikely to occur for certain deterioration modes. This assertion is based on analysis of the results of applying RDA to a wide range of rockslopes in the UK. This data is reported and discussed in Chapter Nine. The treatment measures suggested are cumulative for each class. For example, treatment measures suggested for class 4 stone ravelling should be undertaken, as necessary, *in addition* to measures deemed necessary for classes 1, 2, and 3. In any class, it will usually be unnecessary to apply all of the treatment measures listed, but with engineering judgement and a good understanding of site conditions, to make an appropriate selection.

In addition to treatment measures associated with deterioration of the rockslope, maintenance and management might be required for the stabilisation works themselves. For example, rockbolts might need monitoring, repairing or replacement, planted vegetation might require management (eg thinning, pruning, re-seeding) and masonry walls might require re-pointing. Drainage channels will also need regular clearance and any leaks identified need to be repaired.

As indicated in 8.8.3, the general treatment categories (passive, semi-active etc) are not intended to be rigid and this is reflected in the treatment matrix given in Table 8.3. In some cases, for example, it might be appropriate to adopt retention and support measures (active intervention – class 4) for a class 3 slope, or to use lesser measures for a higher class.

Deterioration mode	RDA$_A$ Class				
	1	2	3	4	5
Grain ravelling	Unlikely to occur	Collector drain at crest; diversion drain at toe; debris clearance (eg from clogged toe drains).	Local reinforced shotcrete application; local mortar screeding; sealing of erosion and fracture chutes; low growing woody vegetation cover at the slope foot.	Widespread shotcrete or dense vegetation cover to reinforce; limited grascrete or crib wall retention.	Unlikely to occur
Stone ravelling	Occasional debris clearance and inspection.	Cut-off drain at crest; rocktrap ditch and fencing; regular scaling and debris and vegetation clearance; local wire netting.	Extensive reinforced wire netting; shotcrete or dentition for small, weak areas; masonry support of small overhangs; weepholes for all surface cover.	Inclined drainage; extensive shotcrete, dentition, underpinning or buttressing as needed (with drains); gabions for weak areas	Benching with intermediate berms; re-excavation if loose areas due to blast damage
Block ravelling	Unlikely to occur	Cut-off drain at crest; regular inspection; rocktrap ditch and anchored or reinforced fencing or barrier; regular scaling and debris and vegetation clearance; limited, high strength wire netting.	Extensive reinforced high strength netting; dentition or masonry walls for weak areas; underpinning of overhangs; weepholes for all surface cover; dowels, bolts or cable lashing for large blocks.	Inclined drainage; underpinning and local buttressing as necessary (eg concrete or masonry wall with drainage layer); gabion retention for large loose areas; rockfall shelter; warning signs and restricted access.	Unlikely to occur
Flaking	Unlikely to occur	Frequent debris clearance; cut-off drain at crest; diversion drain at toe; rocktrap ditch; very close mesh wire or plastic netting.	Surface protection with geotextiles; widespread vegetation cover (grasses and herbs) in suitable materials; large rocktrap ditch and close mesh fence or protective barrier; seal erosion and fracture chutes.	Retention with gabion baskets, crib blocks or grascrete.	Standoff area increased or slope gradient reduced.
Wash erosion	Cut-off drain at crest; diversion drain at toe; regular drain clearance.	Regularly spaced weepholes or inclined drains; local surface protection with geotextile membrane for shallow gradients and vegetation cover in suitable materials	Slope drainage; widespread surface protection with geotextiles and vegetation (grasses and herbs); seal erosion and fracture chutes; shotcrete for local weak areas; dentition with weepholes for local cavities and overhangs; low growing woody vegetation cover at the slope foot.	Surface protection and retention with grascrete (planar slopes only); cross flowpath barriers; underpinning for areas with gullies.	Slope angle and length reduced; benching.
Solution & karstification	Collector drainage at crest.	Regular inspection and removal of vegetation.	Slope drainage; seal and drain vertical fractures and chutes; mortar screeding or dentition of small cavities.	Underpinning of large solution cavities.	Grout infilling of large solution cavities.
Flexural toppling	Unlikely to occur	Infrequent inspection.	Long term movement monitoring; seal and drain major vertical fractures.	Bolting or anchorage of individual key blocks; concrete, masonry or gabion retaining walls.	Unlikely to occur
Grainfall	Unlikely to occur	No action necessary.	Local surface protection with geotextile matting or shotcrete.	Limited dentition; sealing of chutes; inclined drainage.	Unlikely to occur
Stonefall	Infrequent inspection.	Scaling of loose blocks; rocktrap fencing.	Dentition and local underpinning for small cavities and overhangs; rocktrap ditch; low growing woody vegetation cover at the slope foot.	Local buttressing.	Unlikely to occur
Blockfall	Infrequent inspection.	Scaling of loose blocks; rocktrap ditch and reinforced fencing.	Dentition and local underpinning for cavities and overhangs; dowels, bolts or cable lashing for large blocks.	Underpinning of large overhangs; local buttressing; warning signs and restricted access.	Unlikely to occur
Contour scaling	Infrequent inspection.	Removal of loose blocks.	Surface protection with shotcrete or geotextile; containment with wire mesh netting; low growing woody vegetation at the slope foot or rocktrap fencing.	Inclined drainage; extensive shotcrete application.	Unlikely to occur
Slabfall and toppling	Infrequent inspection.	Scaling of loose blocks; rocktrap ditch and reinforced fencing; Movement monitoring.	Dentition and local underpinning for cavities and overhangs; bolts or cable lashing for key blocks; seal vertical fractures.	Underpinning of large overhangs; buttressing; cable reinforcement for potential topples.	Unlikely to occur
Rockfall	Unlikely to occur	Cut-off drains at the crest; diversion drainage at the toe; inclined drains at critical slope locations; movement monitoring; regular inspection; removal of loose blocks; rocktrap ditch and fence.	Reinforcement of key blocks with dowels; local underpinning or buttressing; sealing of vertical fractures at the rear of the slope; warning signs and restricted access.	Substantial retention (eg crib and gabion walls) and local buttressing; reinforced underpinning of overhangs; rockfall shelter.	Re-excavation of slope, removing all loose material back to the new surface.
Debris flow	Unlikely to occur	Cut-off drainage at crest; diversion drainage at toe; inclined drains at critical slope locations; regular drain and ditch clearance; rocktrap ditch and barrier.	For small constituent size: wire mesh netting; local shotcrete; surface cover with geotextiles or vegetation for suitable materials. For large constituent size: dowel reinforcement of key blocks; local underpinning or buttressing; seal vertical fractures at the rear of the slope.	Retaining walls (eg crib blocks and gabion baskets).	Re-excavate slope, scale back to new surface; reduce slope angle and length; cross flowpath barriers.
Rockslide	Movement monitoring.	Inclined drainage.	Anchor or bolt reinforcement for key blocks; seal critical fractures; toe buttressing or gravity retention.	Unlikely to occur	Unlikely to occur

Table 8.3 Treatment measures matrix for different deterioration modes and levels of risk.

CHAPTER NINE
APPLICATION AND VERIFICATION OF RDA

9.1 Application of RDA: Results from the UK

In this chapter the aim is to present the results of applying RDA to the 210 slope units referred to in Chapter Seven (section 7.2.1) and to consider some of the issues pertaining to use of the classification in engineering practice. In the first part of the chapter, the frequency occurrence of RDA classes is described for the slopes investigated in relation to rock mass type and deterioration mode. In section 9.2 a comparison is made between RDA Rating and the Rock Mass Rating (Bieniawski 1979). In section 9.3, three worked examples are provided, one from each of the major types of rock mass (massive, blocky and layered). The chapter closes with a consideration of the applications of RDA, procedures, and issues concerning training and reproducibility.

9.1.1 RDA ratings and adjustments for a range of UK rockslopes

From the field application of RDA it is apparent that the full range of RDA_U classes is represented, with the exception that no rockslopes achieved a rating in excess of 90. The frequency distribution of adjusted and unadjusted RDA ratings is given in Figure 9.1. The data

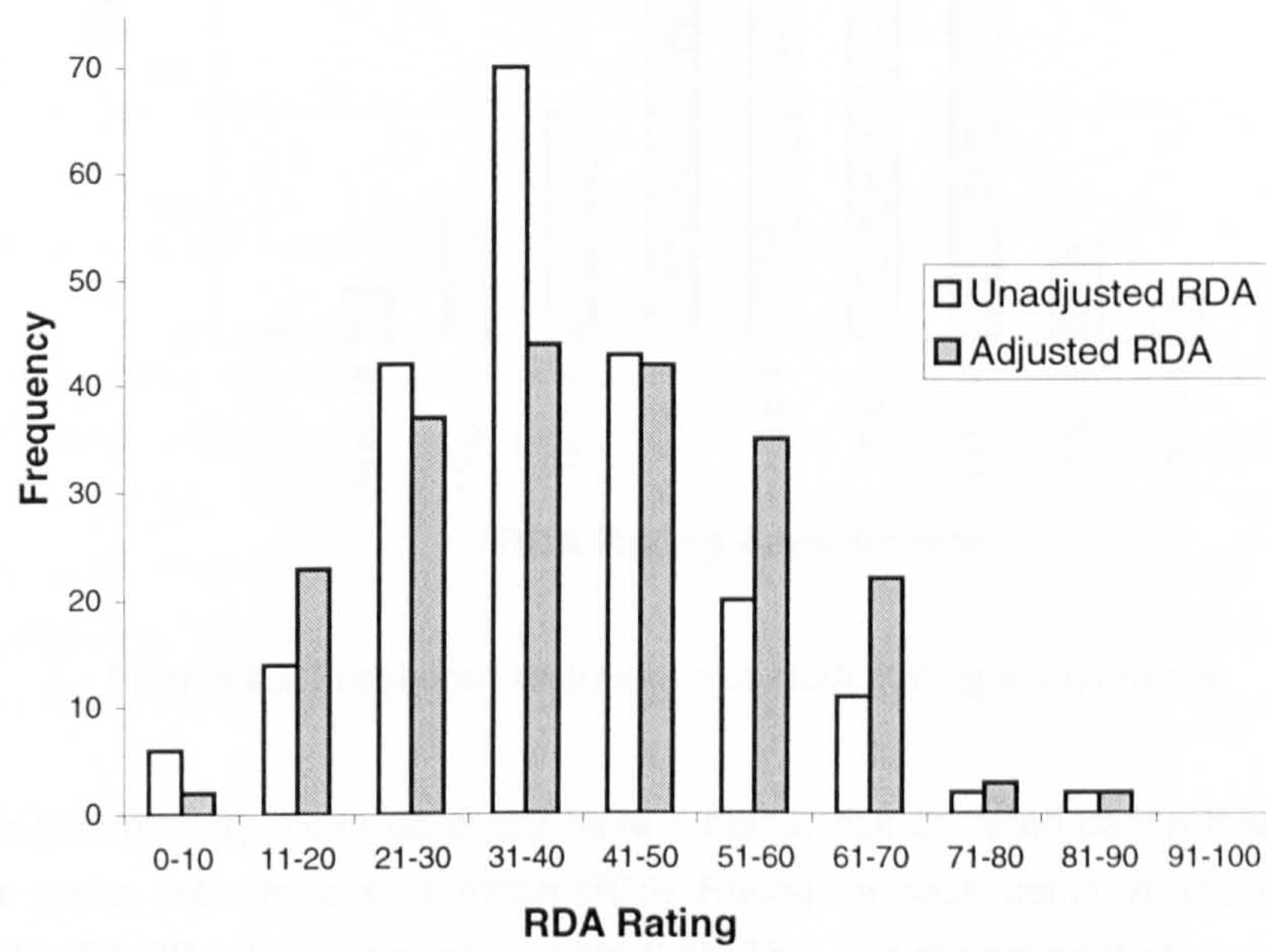


Figure 9.1 Distribution of RDA classes in the case histories investigated

indicate that RDA_U ratings based only on the four key parameters (fracture spacing and aperture, material weathering grade and rock strength) have a distribution with a pronounced peak in the 31 to 40 range with a relatively high frequency of slopes in the lower risk categories. After application of adjustments for external factors, the peak frequency of RDA_A is considerably reduced. There is a general shift in deterioration towards the higher risk ratings. The exception to this is in the 11 to 20 range where rating adjustment produces a significant increase in this low risk group. It is notable that this group includes a large number of slopes where a negative adjustment was applied to account for time since excavation (adjustment L). The rock mass and material properties would have given many of these slopes an RDA_U Rating of 20 to 40, over-estimating the likely deterioration risk. The passage of time (over 100 years in a number of cases) means that many of the slopes have reached equilibrium with their environment and are not deteriorating significantly.

These general results are not unexpected since many of the external factors under consideration have a deleterious influence on deterioration susceptibility. It is expected, therefore, that RDA_A will commonly be greater than RDA_U. This is indicated in Figure 9.2 which shows the frequency distribution of rating adjustments applied. There is a distinct skew in the graph towards low to medium positive adjustments, though adjustments range from -25 to +24. The values for the data collected, and the ratings applied, are given in Appendix 9.A. A summary of ratings, and adjustments applied to rock mass structure properties is given in Table 9.1.

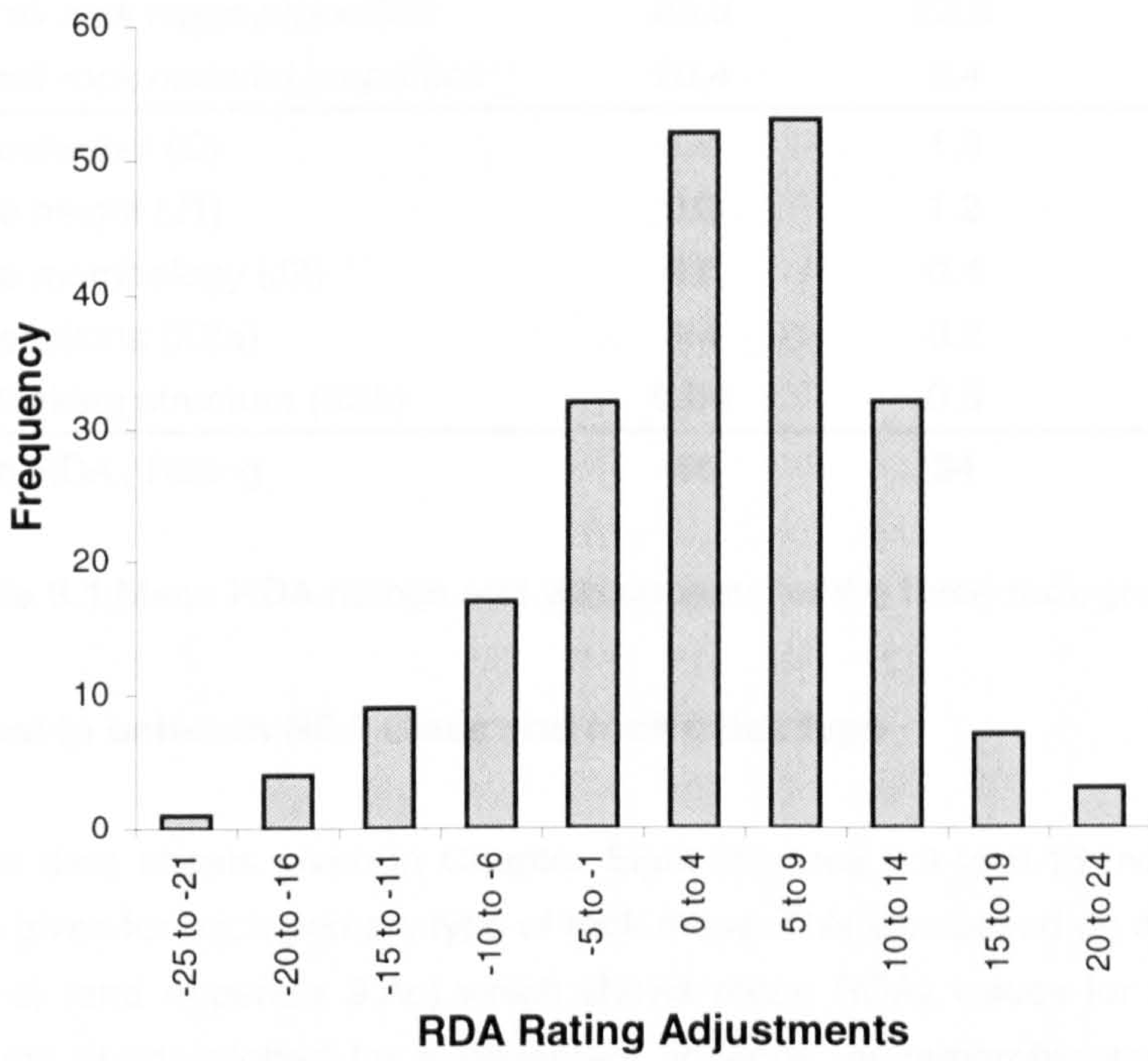


Figure 9.2 Frequency distribution of RDA Rating adjustments

Slopes cut in sedimentary rocks generally have a higher risk of deterioration than igneous and metamorphic rocks, with the overall mean RDA_A Rating for each being 46 (43 RDA_U), 34 (29 RDA_U) and 36 (34 RDA_U) respectively (Table 9.1). This is not unexpected since sedimentary rocks are generally weaker than their igneous and metamorphic counterparts and thus more easily fractured and weathered. To illustrate this, the mean rating for material properties for sedimentary rocks was 20, compared with 6 and 9 for igneous and metamorphic rocks. The respective ratings for mass properties are 23, 22 and 25, indicating much less contrast between rock groups.

Patterns between rock groups are also apparent for some of the adjustment factors relating to some intrinsic properties. For example, adjustments for slope height are greater in igneous and metamorphic rocks than for sedimentary rocks. While this is partly a function of slope design at each locality, it might also reflect the fact that slopes in igneous and metamorphic rocks are more capable of standing at greater heights. Vegetation cover, which was also greater for sedimentary slopes, presumably reflects the fact that it is less successful in colonising tough igneous and metamorphic rocks. Some of the adjustments apply much more commonly to one rock group than another. For example, the incidence of interlocking structure (adjustment K2.b) is extremely rare in sedimentary rocks, while negative adjustments for very regular structure (adjustment K2.a) are high for both sedimentary and metamorphic rocks (Table 9.1).

	Property	Sedimentary	Igneous	Metamorphic
Ratings	Fracture spacing	17.9	17.5	21.9
	Fracture aperture	5.0	5.0	3.0
	Rock strength	15.2	4.0	6.3
	Weathering grade	5.2	2.3	3.0
	Overall rock mass properties	23.0	22.5	24.9
	Overall rock material properties	20.4	6.4	9.3
Adjustments	Groundwater (C)	1.7	1.8	1.4
	Slope height (J1)	0.3	1.2	1.1
	Slope morphology (J2)	0.6	0.4	0.6
	Intersections (K2a)	1.4	0.2	1.4
	Interlocking structure (K2b)	0.04	0.5	0.4
	Mean RDA_A Rating	**46**	**34**	**36**

Table 9.1 Mean RDA ratings and adjustments for the three rock groups

9.1.2 Relationship between RDA Class and rock mass type

In the rock mass data sheets given in Chapter Eight (Figures 8.9 to 8.16 inclusive) a typical RDA_A Class was given for each primary type of rock mass. This was based on data presented in Figure 9.3 (a to d) (and Appendix 9.Ac) which shows mean RDA_A values for each rock mass type, with separate charts plotted for sedimentary, igneous, metamorphic and all rocks. The same data are presented as frequency distribution charts for each rock mass type in Figure 9.4 (a to x). In each chart the number given in parentheses is the mean rating for these slopes.

The charts show that some rock mass types present a low deterioration risk regardless of rock group. This is true for strong massive slopes (Figure 9.3). Equally, fissile and composite (layered) slopes present a high risk of deterioration regardless of rock group. Some types of rock mass are more variable in the deterioration risk which they present, such as layered slopes. These constitute the highest risk slopes in igneous rocks (though are small in number) but are less distinctive for other groups, although the mean RDA_A is not significantly different in each case (Figure 9.3). Considering the frequency distributions (Figure 9.4a to x), some of the charts show a pronounced peak frequency suggesting that a relatively narrow set of conditions or characteristics define the rock mass type. This is well illustrated by comparing the distributions for strong massive slopes (Figure 9.4a to d or Figure 9.4m to p) with those for fissile rock masses (Figure 9.4q to s). Strong massive slopes are characterised by a dominant frequency in classes 1 and 2. As might be expected, the weaker sedimentary rocks dominate class 2 whereas metamorphic rocks dominate class 1. In contrast, the deterioration risk associated with fissile rock masses not only differs significantly between sedimentary and metamorphic rock groups, but overall, also has a very wide distribution with no peak apparent. This reflects the fact that in sedimentary rocks, fissile rock masses tend to be highly weathered, extremely weak, closely laminated shales and mudstones. Metamorphic fissile rockslopes include some similarly weak materials, but also include strong, competent rock masses in metasediments where

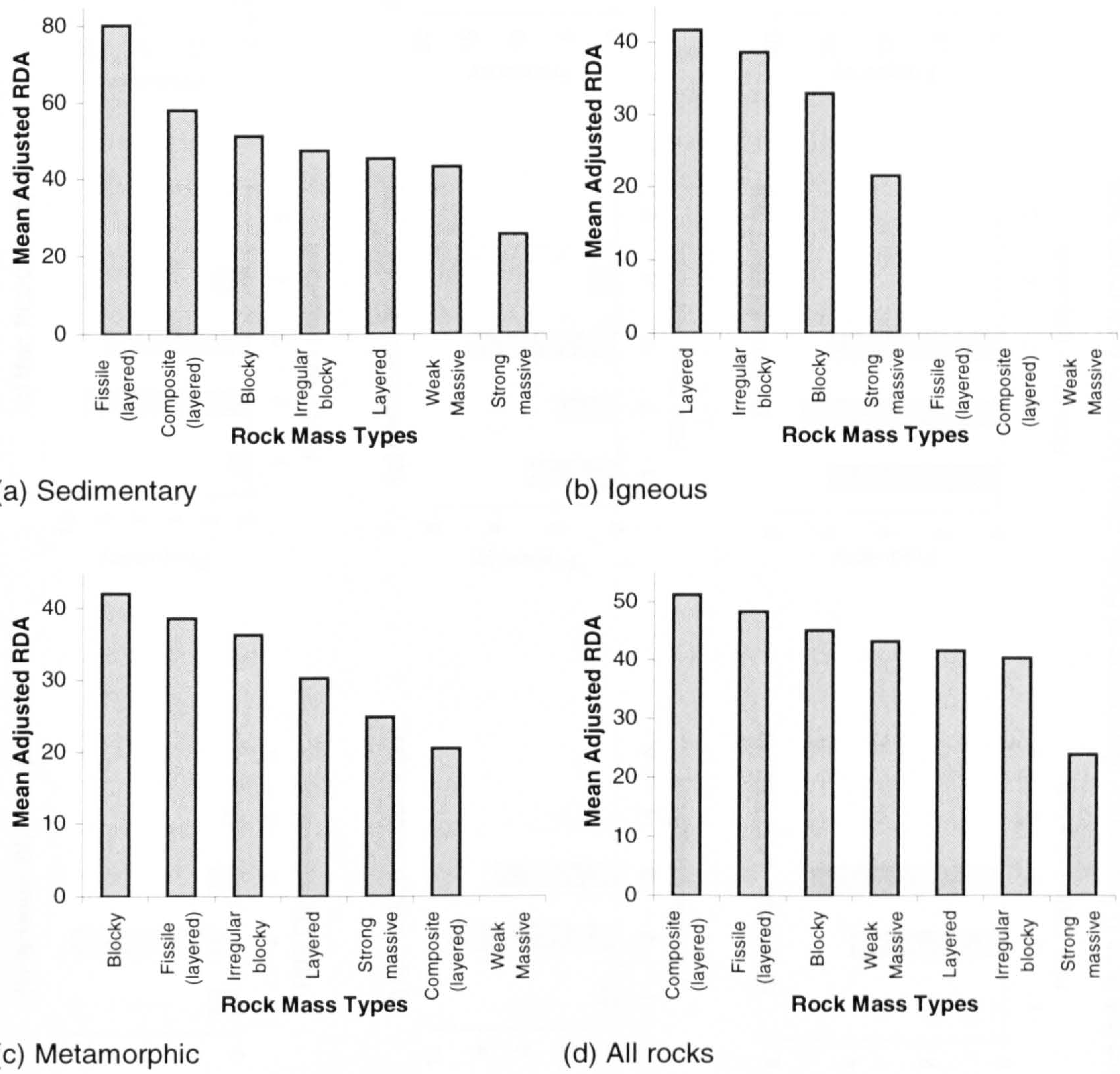


Figure 9.3 (a to d) Mean RDA_A ratings for each rock mass type

fissility relates to strong cleavage. In the case of blocky rockslopes, peak frequencies occur in RDA_A classes 2 and 3 giving a distinctively peaked distribution for all rocks. Blocky rockslopes occur across the full range approximately in proportion to the frequency of slope numbers in the RDA Classes (Figure 9.1), though examination of the distribution for sedimentary, igneous and metamorphic rocks shows there is variation between rock groups, with blocky rock masses occurring more commonly in the weaker classes for sedimentary rocks. This distribution reflects the fact that in order to be defined as blocky, a rock mass must be moderately fractured and is therefore unlikely to be extremely or very strong. Thus the overall mean RDA_A Rating of 45 represents the middle of the range of measured properties.

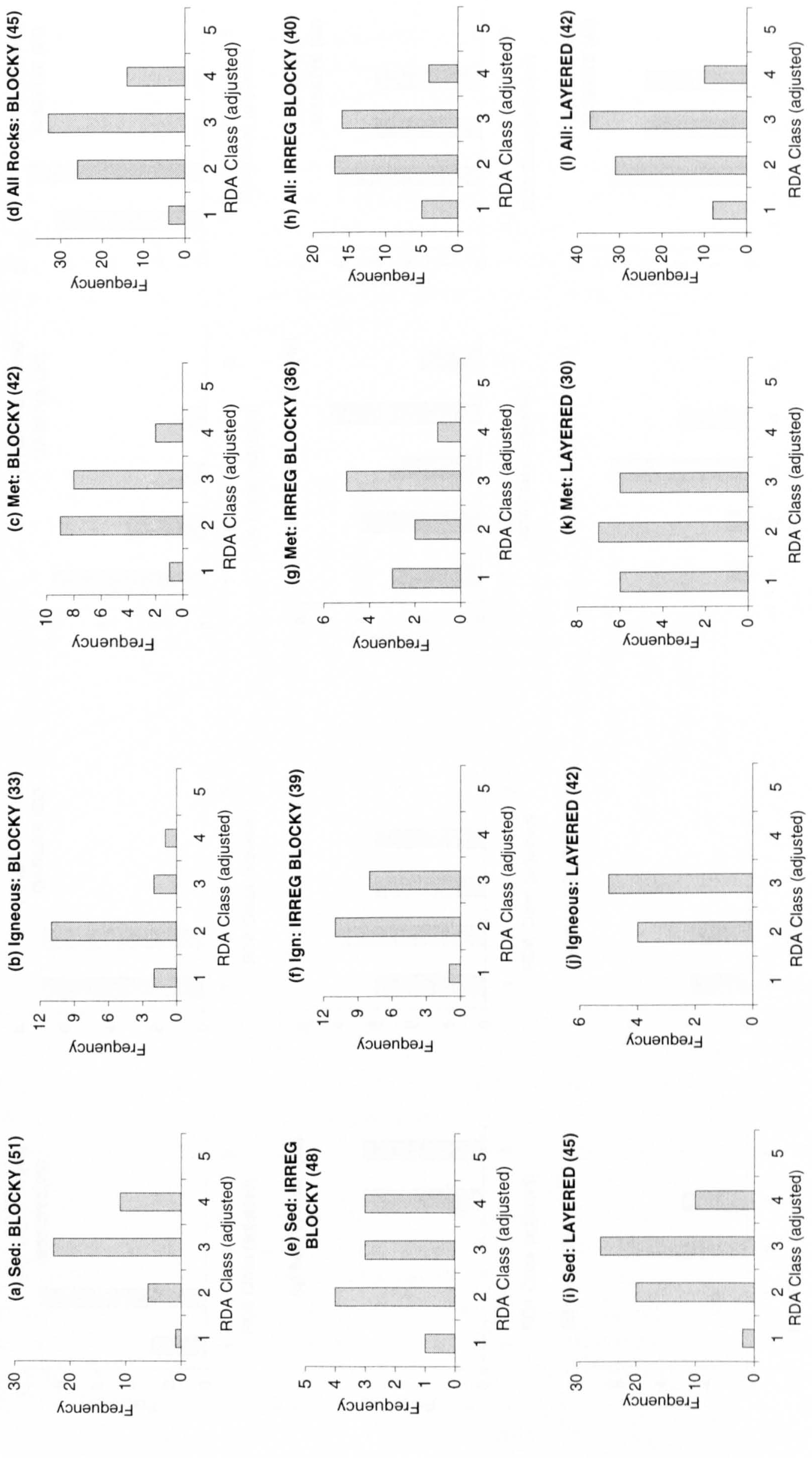


Figure 9.4 (a to l) Frequency distribution of RDA_A class for each rock mass type

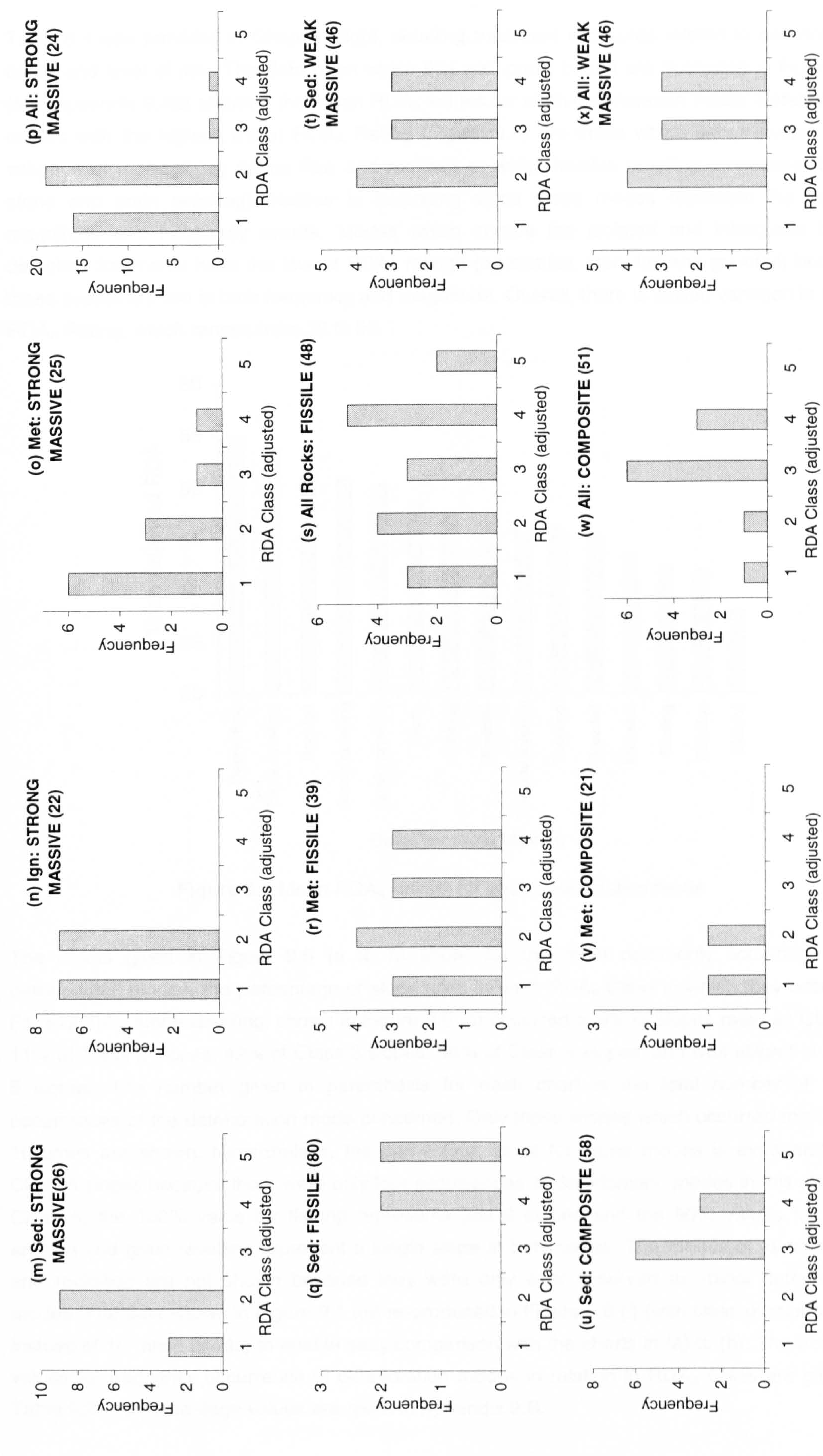


Figure 9.4 (m to x) Frequency distribution of RDA_A class for each rock mass type

9.1.3 Relationship between RDA Class and deterioration mode

Table 8.3 was provided in Chapter Eight, detailing treatment measures related to deterioration mode and level of risk. The data upon which this was partly based are illustrated in Figure 9.5 (and Appendix 9.Ac) showing the mean RDA_A values for each deterioration mode. Deterioration modes with the highest mean RDA_A Rating (Figure 9.5) are those which either involve large volumes of material (eg debris flow and rockfall) or which involve ravelling processes (block, stone and grain ravelling). Neither is surprising since these modes represent the highest magnitude and frequency events. Modes which involve the isolated and infrequent fall of detached fragments have the lowest RDA_A ratings (eg slabfall, stonefall and grainfall) because these events are low in both frequency and magnitude. Overall, there is limited variation in mean RDA_A Rating, which ranges from 38 to 55.

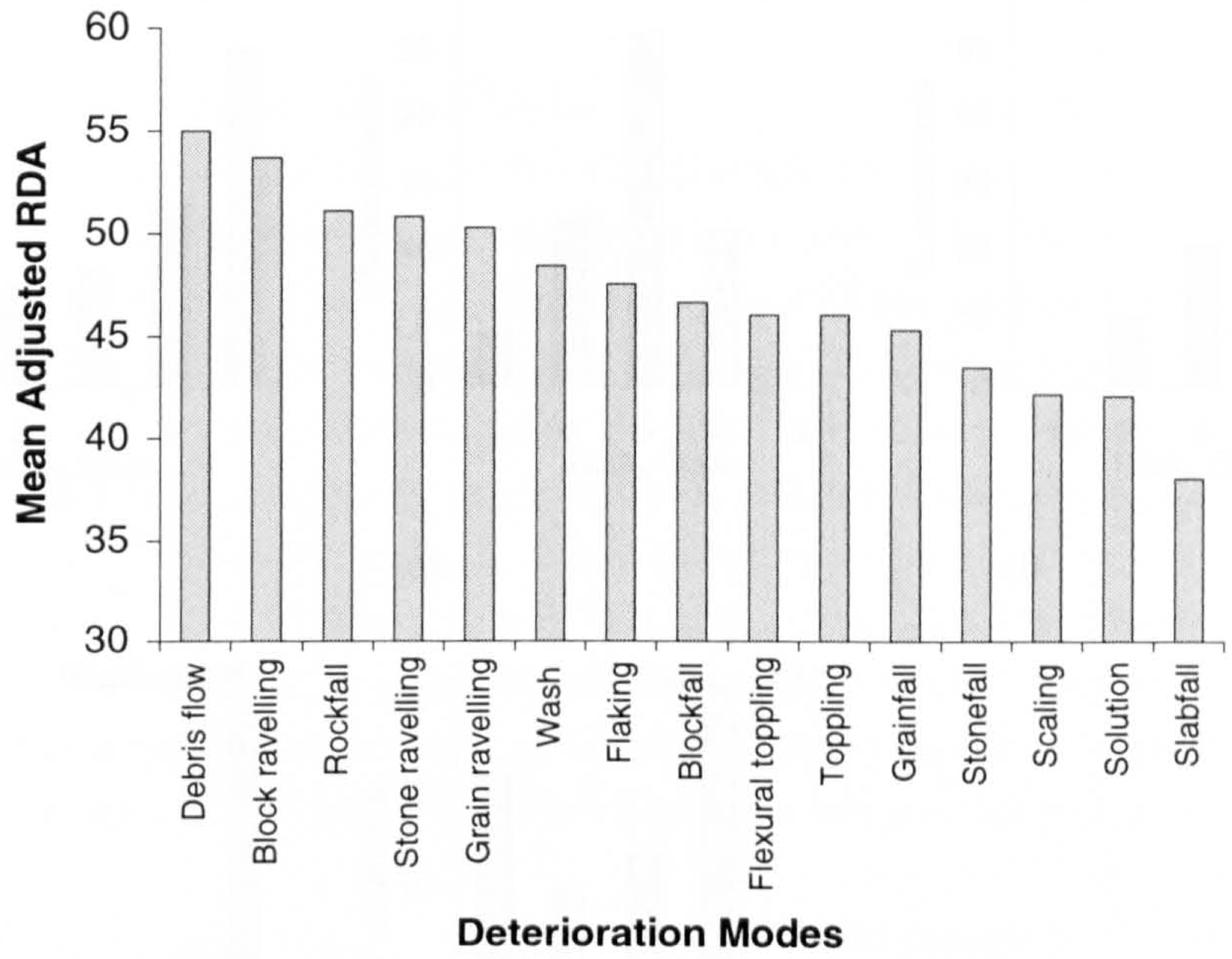
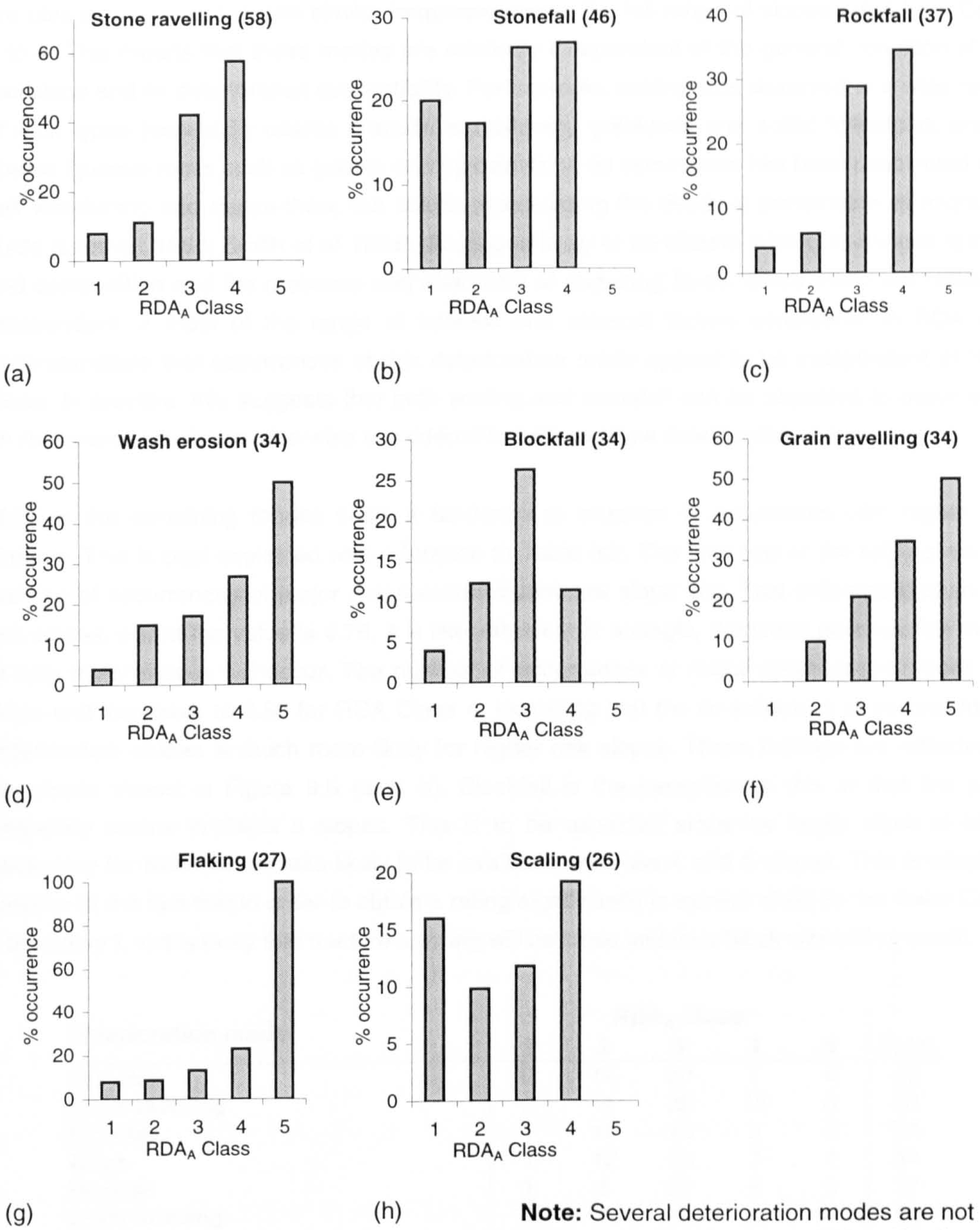
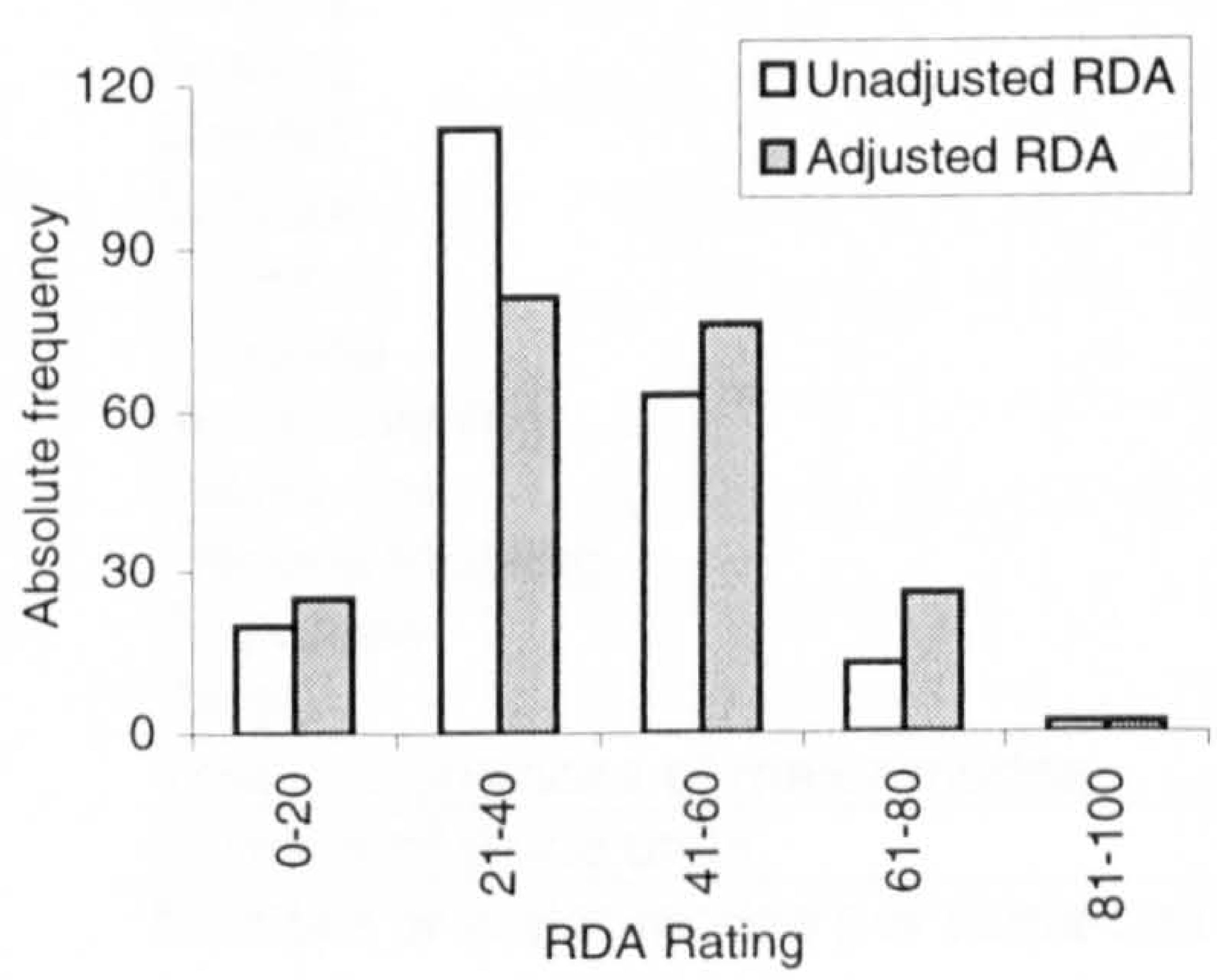


Figure 9.5 Mean RDA_A ratings for each deterioration mode

The charts given in Figure 9.6 (a to h) show, for the most commonly occurring *major* deterioration modes, the percentage of slope units in each RDA_A Class in which they occurred. For example, stone ravelling, shown in Figure 9.6 (a) occurred in 8% of slopes rated as Class 1, 11% of Class 2 slopes, 42% of Class 3 slopes, 58% of Class 4 slopes, and was absent in Class 5 slopes. The number given in parenthesis for each chart is the total number of major occurrences of the deterioration mode concerned. Only those modes which occurred more than 10 times are shown. Nevertheless, the percentage value for some modes is exaggerated in Class 5 slopes because there were only four occurrences of deterioration modes in this class. In Class 5, the 100% value for flaking represents just 2 slopes and the 50% values for wash erosion and grain ravelling represent a single slope in both cases. The modes of karstification and rockslide are not shown because they were only ever observed as *minor* deterioration modes. The data shown in Figure 9.1 are re-produced in Figure 9.6 (i) (with class divisions of 20 instead of 10 rating points) to enable easy comparison with the charts in (a) to (h). The absolute values for frequency occurrence of deterioration modes in relation to RDA_A Class are given in Table 9.2 and percentage values are given in Appendix 9.B.





Note: Several deterioration modes are not shown in the charts provided: Karstification and rockslides were never observed to be a *major* mode, and the total number of occurrences of other modes was less than 10 in each case. The number given in parenthesis is the total number of *major* occurrences for each deterioration mode.

Figure 9.6 (i) (*left*) shows the overall frequency distribution of adjusted and unadjusted RDA classes (compared with Figure 9.1 which gives the distribution in divisions of 10 rating points).

Figure 9.6 (a to i) The occurrence of deterioration modes as a percentage of slopes in each class

The charts in Figure 9.6 show two distinctive distribution patterns. Stonefall and scaling (b and h) are ubiquitous, occurring with similar frequency across the full range of slopes rated from Class 1 to 4. This means that these modes are relatively independent of the general condition of the rockslope and its deterioration susceptibility. For example, scaling was observed in a wide range of rock types (especially coarse granular sandstones, gritstones and oolitic limestone, and in coarse igneous rocks such as granite and pyroclastics). Its occurrence has been associated with salt weathering and freeze-thaw, the scales representing the depth of penetration of migrating fluids (Lienhart 1988; Smith et al 1994). Scaling is likely to be closely related to mineral texture and composition and the presence and chemistry of migrating fluids. Since these are relatively independent of most of the range of intrinsic and external factors considered in RDA it is understandable that occurrences of this deterioration mode appear to be independent of RDA Class. In practice, this suggests that both scaling and stonefall can be expected to occur even on rockslopes which are otherwise considered to be a very low deterioration risk.

Most of the remaining modes show a tendency to increase in occurrence with higher risk classes. This is best explained with reference to Table 9.2. The final row of the table gives the number of occurrences of major deterioration modes per slope unit. This indicates that on low risk slopes, where the value is 0.76, it is likely that either a single, dominant deterioration mode or only minor modes will occur. The number of occurrences of major deterioration modes per slope unit increases to 2.58 for RDA Class 4, indicating that the co-existence of several major deterioration modes is much more likely for higher risk slopes. These findings are reflected in the charts shown in Figure 9.6 (a to h). Blockfall is the exception to this in that the peak frequency occurs in Class 3 slopes. This is to be expected since the larger sizes of block necessary for blockfall are less likely to be available on Class 4 and 5 slopes. This is simply a function of the fact that in order to obtain a rating significantly in excess of 60 (ie the lower Class 4 boundary), is it is likely that fracture spacing will be close and thus block size will be small.

Deterioration mode	**RDA$_A$ Class**					
	1	**2**	**3**	**4**	**5**	**Total**
Stonefall	5	14	20	7	0	46
Stone ravelling	2	9	32	15	0	58
Blockfall	1	10	20	3	0	34
Wash	1	12	13	7	1	34
Rockfall	1	5	22	9	0	37
Grain ravelling	0	8	16	9	1	34
Flaking	2	7	10	6	2	27
Scaling	4	8	9	5	0	26
Slabfall	1	3	5	0	0	9
Solution	1	3	0	2	0	6
Grainfall	1	2	3	3	0	9
Toppling	0	1	2	0	0	3
Block ravelling	0	0	5	1	0	6
Debris flow	0	0	3	0	0	3
Flexural toppling	0	0	1	0	0	1
Rockslide	0	0	0	0	0	0
Karst	0	0	0	0	0	0
Total occurrences of major modes	19	82	161	67	4	333
Number of slope units	25	81	76	26	2	210
Number of major modes per slope unit	**0.76**	**1.01**	**2.12**	**2.58**	**2.0**	

Table 9.2 Occurrence of deterioration modes in relation to RDA$_A$ Class

9.2 Comparison of RDA Stage One with Rock Mass Rating (RMR)

Given the widespread usage of the Rock Mass Rating (RMR) system (Bieniawski 1979) in rock engineering, and the fact that there is some similarity in general form between the RMR and Stage One of RDA, it is useful to determine any empirical relationship between them. The ratings used in the RMR are given as Appendix 9.C (after Bieniawski 1979). Empirical relations between RMR and a variety of other classifications, including the Q system (Barton et al 1974), the Rock Structure Rating (RSR) concept (Wickham et al 1972) and the Geological Strength Index (GSI) (Hoek 1994), have been established:

RMR = 9 ln Q + 44 (Bieniawski 1976) [9.1]

RSR = 0.77 RMR + 12.4 (Rutledge and Preston 1978) [9.2]

For RMR > 23: GSI = RMR – 5 (Hoek 1994) [9.3]

Figure 9.7 (a to d) shows plots of unadjusted RDA ratings against RMR values, also unadjusted for slope favourability. This is to ensure comparison of like with like. With a regression coefficient of 0.79, the overall relationship (Figure 9.7d) is

RDA = -0.85 RMR + 89.07 [9.4]

though there is a significant amount of scatter. However, this general correlation disguises a significant difference in relationship between the two ratings for igneous rocks, where the regression coefficient is just 0.48, and the equation notably different from the other rock groups. The shallower gradient of the trendline indicates that the RMR does not make as significant a distinction between igneous rockslopes as does RDA. This is because the RMR ratings tend to emphasise the major joint sets, which in igneous rocks, are particularly widely spaced. RDA tends to emphasise the small block sizes determined by stress release, blasting and weathering-related fractures. The contrast between these two types of fractures is much greater in igneous rocks which partly explains why the correlation for this rock group differs from the others.

Comparison of the two ratings given in Figure 9.7d shows that RMR data points are spread over a smaller range of values (eg 26 to 86) than those for RDA (eg 4 to 85). This means that RDA provides greater distinction between slopes in the middle part of the range. This is useful for evaluating treatment requirements since it allows for greater discrimination between the most commonly occurring range of slopes. While both ratings fully utilise the range of values available for high quality rock masses, RDA provides greater discrimination between low quality rockslopes. This is illustrated by the fact that using the regression equation, a slope with an RDA_U Rating of 5 (indicating extremely low deterioration risk) would obtain an RMR rating of 99, just below the maximum rating possible. Conversely, a slope with an RMR rating of 5 (indicating an extremely low quality rock mass) would obtain an RDA_U Rating of 85, which is only just into the very high risk class. This is useful for discriminating between treatment requirements for rockslopes which are likely to have the greatest need for mitigation. That there is a moderate scatter of points either side of the regression lines in Figure 9.7 is not unexpected given the different basis for calculation of the two ratings schemes: The variation in properties considered in each classification system obviously contributes to this (RMR considers discontinuity spacing; RQD; discontinuity condition; groundwater seepage and rock strength) as does the different weighting applied to these properties. However, the emphasis in RDA on inclusion of *all* fractures regardless of origin, is probably the most significant factor.

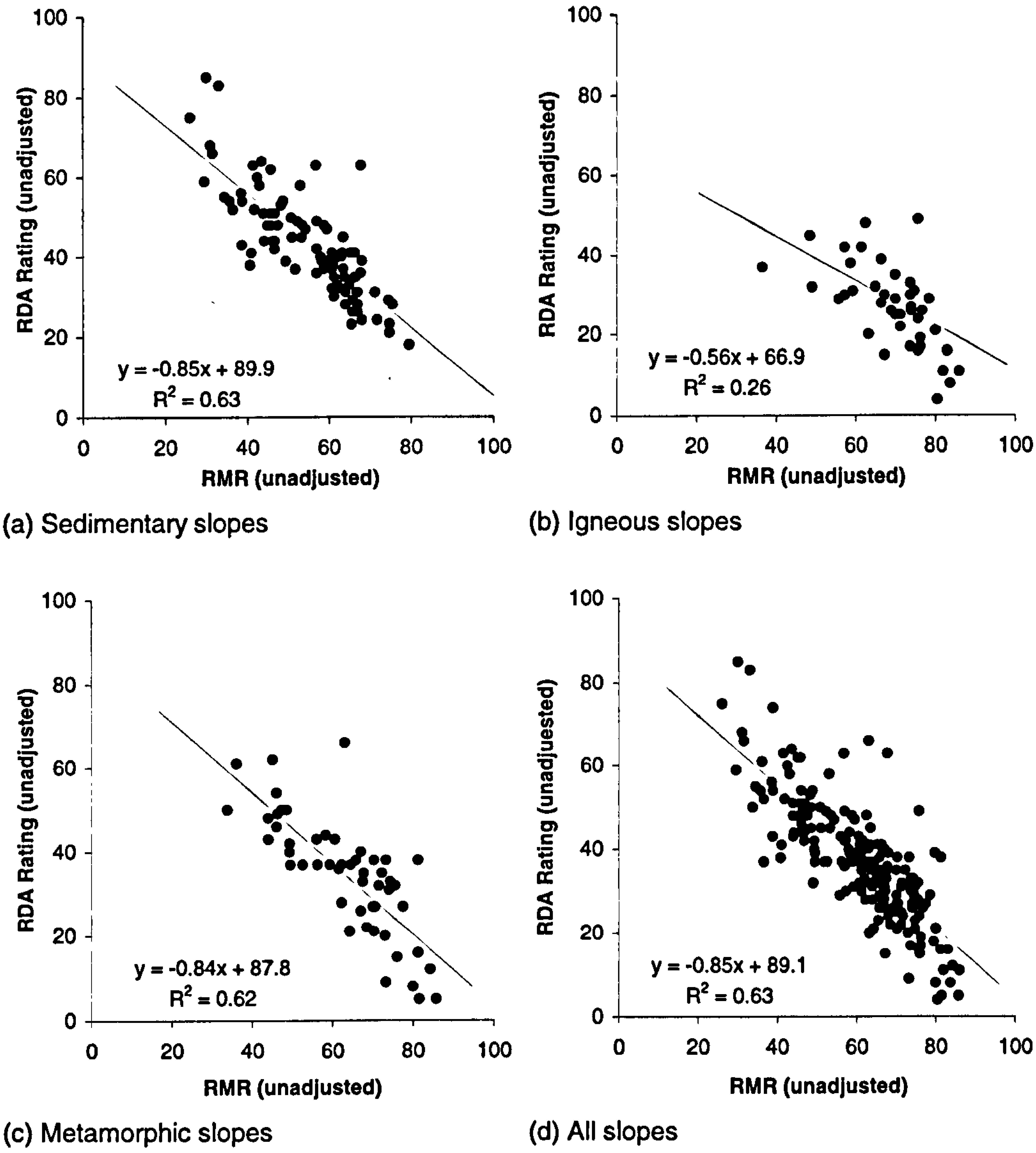


Figure 9.7 (a to d) Scattergraphs of RDA_U against RMR

9.3 Selected Rockslope Deterioration Assessments

In this section, three worked examples of RDA are presented. The first is a weak massive rockslope in sandstone, the second is an irregular blocky rockslope in ignimbrite, and the third is a fissile and layered rockslope in metasediment.

9.3.1 Bongate Scar: Weak massive rockslope

Bongate Scar is a very old, hand excavated quarry situated at the edge of the floodplain of the River Eden in Cumbria at grid reference NY 687199 (Plate 9.1). The rock is a medium grained,

Plate 9.1 General view of Bongate Scar

uncemented, friable, slightly weathered, weak SANDSTONE with characteristic red coloration due to haematite coating of grains. The rocks are part of the Penrith Sandstone formation of Permian age. The structure consists of very large scale dune cross bedding with occasional vertical joints. The slope is mostly uniform, but contains some very localised intensely fractured zones. For clarity, the latter are not considered in the RDA presented here.

9.3.1.1 Stage One RDA

The unadjusted RDA_U Rating for the slope is 41 and was calculated as follows:

Key parameter	Value	Rating
Fracture spacing	>250cm	0
Fracture aperture	2mm	6
Rock strength	4MPa	32
Material weathering grade	Fresh to slightly weathered	3
	Unadjusted RDA Rating (RDA_U)	**41**

Table 9.3 Calculation of RDA_U for Bongate Scar

On this basis, referring to Figure 8.2 and Table 8.2, this slope is classed as being on the borderline between low to moderate risk (class 2/3). Adjustment factors were calculated on the basis of the following information:

Environmental conditions: The slope is located at an altitude of 145mAOD. It has a west to north west aspect and is situated in an extremely sheltered valley bottom environment, surrounded by mature trees. The slope therefore receives little direct sunlight and appears to remain damp for much of the time. A widespread cover of moss and algae on parts of the rock surface is further evidence of this. There is no seepage present but evidence of waterflow exists in the form of closely spaced laminations being picked out by wash erosion (Plate 9.2).

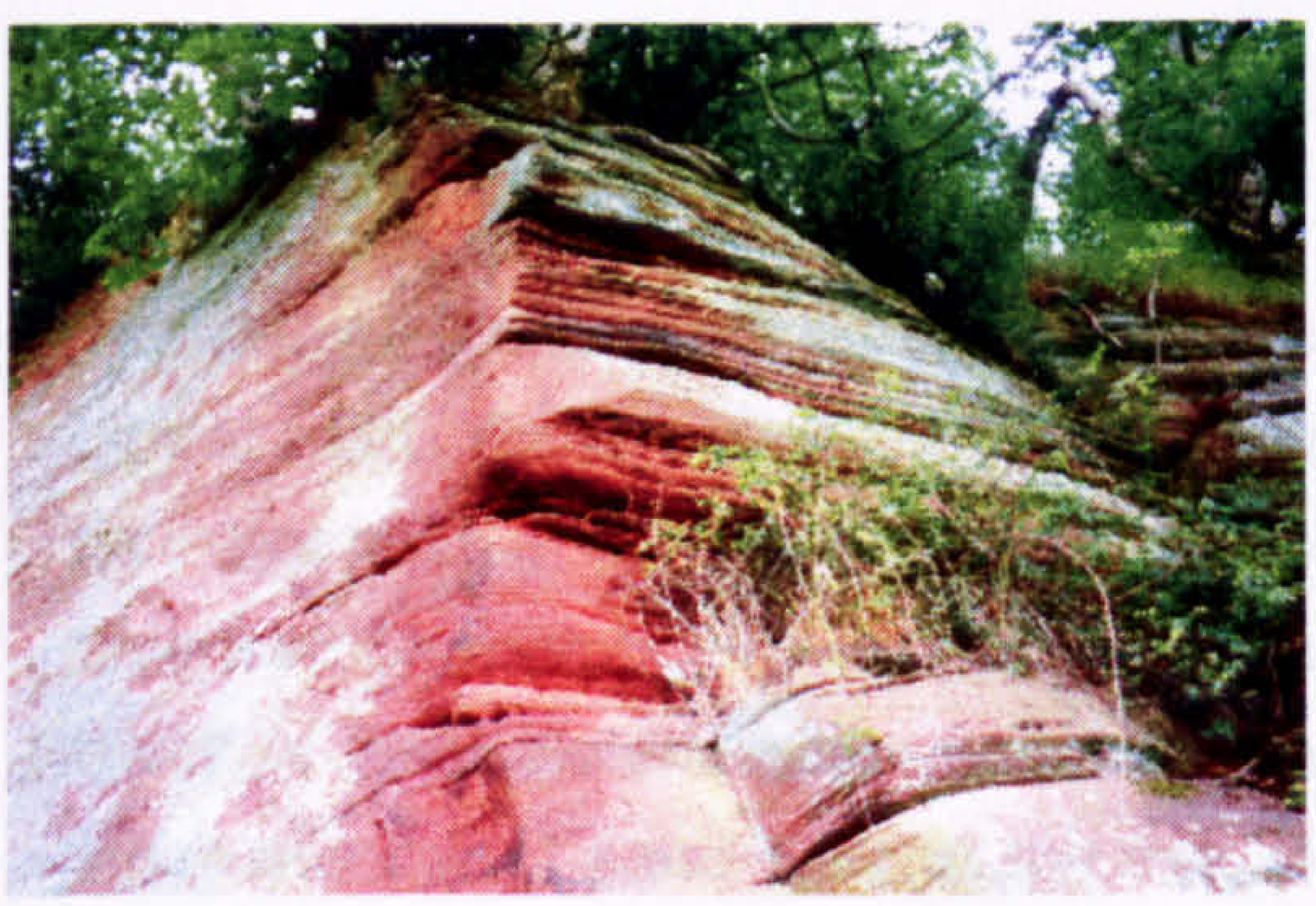


Plate 9.2 Evidence of waterflow at Bongate Scar

Stress conditions: There are several large, mature trees at the edge of the crest of the slope, some of which overhang (Plate 8.5A). The slope is located behind a church in a small village and is not subject to any dynamic stresses from blasting or traffic vibration.

Engineering factors: There is no evidence of any engineering intervention.

Excavated slope characteristics: Apart from the moss and algae already mentioned, the only vegetation *on* the slope face are one or two mature trees which have managed to grow successfully in the widely spaced vertical joints (Plate 8.5A). There is some small scale fragmentation associated with these. The slope reaches a maximum height of 15m but is more typically 12m, and has been cut to a gradient of 85 to 88°. The slope comprises a single lift and is quite planar. Most of the adjustments for rock mass structure do not apply, since the RDA_U does not satisfy the conditions for adjustment K1 (where the total rating for rock mass or rock material properties must be >35), and the rock mass is massive and uniform. The only open fractures present are vertical joints, and so there is no intersection of fractures.

Other adjustments: The quarry has not been excavated for at least 80 years and was cut by hand. Cattle and sheep graze on the floodplain at the foot of the slope but are unlikely to cause any disturbance. There is no evidence to suggest that the slope has been flooded in recent years though it remains a possibility if river discharge were to exceed bankfull.

From the above observations, a total adjustment of -7 was made, calculated as follows:

Code	Description or value	Rating
A2.a	Moisture pocket	1
C1.b	Moderate areas of damp rock	2
D2	Surcharge due to trees	1
H3.c	Local effect of large trees on face	2
J2.b	Uniform, planar slope	-2
K2.a	Lack of fracture intersection	-2
L1.a	>80 years since excavated	-9
	Total adjustment	**-7**

Table 9.4 Total RDA Rating adjustment for Bongate Scar

The adjusted RDA_A Rating, therefore, is 41 + -7 = **34**, which is a class 2, low risk slope, requiring a passive approach to mitigation.

9.3.1.2 Stage Two RDA

The slope fits neatly into the weak massive type. The relevant data sheet (Figure 8.9) indicates a typical RDA_A Class of 2/3+, and associated deterioration modes of grain ravelling and grainfall as major modes, with wash erosion and scaling as minor modes. Stonefall can also occur. Since deterioration morphology is available, analysis of the relevant data sheets will help to confirm which of these deterioration modes is actually present. In practice, field observation showed a small number of sand piles at the foot of the slope, containing occasional flakes or spalls of intact sandstone. This indicates grain ravelling and scaling. There was also a general scatter of sand grains at the foot of the slope indicating grainfall, and a handful of stone-sized fragments indicating stonefall. As mentioned earlier, the slope provided evidence of waterflow over the surface and there was small scale fragmentation around large tree roots. These indicate a possible contribution of wash erosion and the potential activity of localised fragmentation due to root wedging.

9.3.1.3 Stage Three RDA

Reference to the deterioration mode data sheets (Figures 8.18 to 8.28) and Table 8.3 should enable an assessment to be made of the most appropriate mitigation measures for this slope. As stated in Chapter Eight, the mitigation recommendations are based on the presumption that the consequences of deterioration would be unacceptable if not treated. On this basis, the recommended treatment for this slope would be crest and toe surface drainage and very infrequent removal of loose blocks or spalls. In reality, this slope is situated on private land but has been designated a Regionally Important Geological and Geomorphological Site (RIGS). A booklet produced by Cumbria RIGS Group (1994) on the geology of the Eden Valley ensures that this locality is visited frequently by educational groups and individuals. Nevertheless, the deterioration modes involved are unlikely to have any serious consequences. The exception to this is that there is the remote possibility, given the overhanging trees at the crest, for a significant collapse to occur, and thus occasional ongoing monitoring of this situation would be recommended.

9.3.2 Knock Pike Quarry: Irregular blocky rockslope

Knock Pike is a disused, bulk blasted quarry situated near Appleby in Cumbria at grid reference NY 687286. The rock is an ash flow unit with eutaxitic texture, formed from intense compaction of pumice fragments contained within. It is a fine crystalline, well cemented, slightly weathered, very strong IGNIMBRITE. The sequence is of Ordovician age, of the Knock Pike Tuff formation (Borrowdale Volcanic Group). The rock mass is dominated by random, blast-induced fractures and irregular joints, giving an irregular blocky structure overall (Plate 9.3).

9.3.2.1 Stage One RDA

The unadjusted RDA_U Rating for the slope is 49 and was calculated as follows:

Key parameter	Value	Rating
Fracture spacing	15cm	26
Fracture aperture	7mm	12
Rock strength	150MPa	4
Material weathering grade	Slightly weathered, locally moderately weathered	7
	Unadjusted RDA Rating (RDA_U)	**49**

Table 9.5 Calculation of RDA_U for Knock Pike Quarry

On this basis, referring to Figure 8.2 and Table 8.2, this slope is classed as being moderate risk (class 3). Adjustment factors were based on the following information:

Environmental conditions: The slope is located at an altitude of 335m. It has a north westerly aspect and is situated in high, open moorland. However, a vegetated hill opposite the quarry considerably reduces the amount of exposure. There is no direct or indirect evidence of groundwater seepage or regular surface runoff.

Stress conditions: The quarry does not appear to be subject to any dynamic stresses.

Engineering factors: There is no evidence of engineering intervention. The slope was excavated by bulk blasting, producing a set of irregular, closely spaced and wide aperture fractures.

Plate 9.3 Blast induced fractures at Knock Pike Quarry

Excavated slope characteristics: There is an extremely small amount of vegetation on the slope, mainly consisting of a few grasses and herbaceous plants, and there is no deterioration specifically associated with these. The slope is up to 30m high but more typically 25m and has a gradient of 54^{o} from the horizontal. The slope has been cut in a single lift but it has an irregular surface. Three of the adjustments pertaining to rock mass structure apply: K1.c applies because the total rating for mass properties (38) is greater than 35, and the total rating for material properties (11) is less than 30% of the unfavourable parameters. K2.b also applies because the blocks produced by irregular fracturing are highly interlocking (Plate 9.3). A negative adjustment must be made for K3 because although 15cm has been used in the RDA_U to define block size, the slope also includes much larger blocks of 30-50cm in places, though not sufficiently distinct in spatial distribution to zone the slope and apply a separate RDA.

Other adjustments: The age of the quarry is unknown, but judging from the presence and appearance of machinery lying around on the quarry floor, the size of quarry floor vegetation and the state of the access track, it has probably been quarried in the last 20 to 30 years. The quarry is not subject to any direct disturbance.

From the above observations, a total adjustment of +5 was made, calculated as follows:

Code	Description or value	Rating
B1.a/d	North westerly aspect	2
J1.a	Slope height	5
K1.c	Much more favourable material properties than mass properties	5
K2.b	Interlocking structure	-2
K3	Variability	-5
	Total adjustment	**+5**

Table 9.6 Total RDA Rating adjustment for Knock Pike Quarry

The adjusted RDA_A Rating, therefore, is 49 + 5 = **54**, which is a class 3, moderate risk slope, requiring a semi-active approach to mitigation.

9.3.2.2 Stage Two RDA

The slope fits neatly into the irregular blocky type. The relevant data sheet (Figure 8.15) indicates a typical RDA_A Class for igneous rocks of 2/3, and associated deterioration mode stone ravelling as the major mode, with stonefall and rockfall as minor modes. Grain ravelling, flaking, wash erosion, blockfall and rarely, debris flow, might also occur. Since deterioration morphology is available, analysis of the relevant data sheets will help to confirm which of these deterioration modes is actually present. In practice, field observation showed several large overhangs, highly fragmented beneath, and probably the source of large, associated rockpiles from rockfalls (overhang collapse). Irregular structure chutes are also present, feeding large debris piles at the foot, indicating debris flow (Plate 9.4). There are poorly developed levee structures and lobe-like deposits associated with these. The slope also has a number of large, perched blocks indicating potential for stonefall and blockfall, varying length ledges acting as structural chutes, and evidence of block wedging in wide aperture fractures. In some areas, blocks are tightly interlocking, so although closely fractured, they appear stable. Observations of erosional and depositional landforms suggest that while generally stable, the rock mass structure is such that when deterioration by fall is triggered in an area, this releases a large volume of material to collapse also. Thus the principal deterioration modes are stonefall, rockfall and debris flow. There might also be some stone ravelling but evidence is hidden in the irregularity of the slope and the large amount of debris from the large magnitude events.

Plate 9.4 Knock Pike Quarry Structure chutes with debris piles at the foot

9.3.2.3 Stage Three RDA

Reference to the deterioration mode data sheets and Table 8.3 should enable assessment to be made of the most appropriate mitigation measures for this slope. The recommended treatment for this slope would be scaling of perched blocks; protective measures (large rockfall ditch and reinforced or anchored fencing or barrier); regular debris clearance, inspection and monitoring; cut-off drain at the crest; underpinning of overhangs; local dentition or buttressing; and dowel reinforcement of key blocks. Local application of wire netting might also be helpful in areas where only stone ravelling is occurring (ie none of the large magnitude events are in evidence).

The deterioration modes involved here have the potential for serious consequences because of their high magnitude. However, the shallow gradient of the slope ensures that most release of material from the face is dominated by sliding and bouncing modes of transport rather than freefall. In reality also, this slope is situated on private land, though a public footpath passes close to the entrance of the quarry. Since access to the face is extremely difficult, it is unlikely that there is any serious hazard for passers by and so the quarry could be left in its present condition without any mitigation. For minimisation of the potential hazard, a warning sign or access restriction (eg a boundary fence) would be useful.

9.3.3 M6 Dillicar: Fissile and layered/blocky rockslope

Plate 9.5 M6 Dillicar Interbedded competent sandstone and fissile shaley layers

The cutting at Dillicar on the M6 in Cumbria is situated at grid reference NY 610002. The rock is a fine to medium grained, moderately (fissile areas) to strongly (layered areas) cemented, fresh to slightly weathered, strong to very strong METASEDIMENT, consisting of a slightly metamorphosed turbidite sequence with interbedded siltstone and fine sandstone. The sequence is of Upper Silurian age, of the Coniston Grits formation. The structure consists of inter-layered competent sandstones and siltstones, with fissile shaley layers (Plate 9.5). The overall rock mass could therefore be described as composite (layered), but the two zones (fissile and layered) have been evaluated separately.

9.3.3.1 Stage One RDA

The unadjusted RDA_U Rating for the layered slope is 28, and for the fissile slope is 50. Calculations are as follows:

LAYERED/BLOCKY SLOPE		
Key parameter	**Value**	**Rating**
Fracture spacing	35cm	20
Fracture aperture	0.4mm	2
Rock strength	140	4
Material weathering grade	Fresh to very slightly weathered	2
	Unadjusted RDA Rating (RDA_U)	**28**
FISSILE SLOPE		
Fracture spacing	10cm	28
Fracture aperture	1.5mm	5
Rock strength	50	13
Material weathering grade	Fresh to very slightly weathered	4
	Unadjusted RDA Rating (RDA_U)	**50**

Table 9.7 Calculation of RDA_U for M6 Dillicar

On this basis, referring to Figure 8.2 and Table 8.2, the layered/blocky (Figure 8.11 and 8.14) slope is classed as being low risk (class 2) and the fissile slope (Figure 8.12) is classed as being moderate risk (class 3). Adjustment factors were calculated on the basis of the information given below, which pertains to both layered and fissile slopes unless otherwise stated:

Environmental conditions: The cutting is located at an altitude of 200m. It has a north easterly aspect and is extremely exposed, overlooking the Lune Gorge with open moorland across the valley to the east. There are many areas of light seepage flow and areas of general wet rock surface.

Stress conditions: The slope is not subject to any dynamic stress from quarry blasting, and although situated alongside a motorway, is also not subject to traffic vibration. This is because in addition to a grass verge at the foot of the slope which is commonly more than 15m wide, there is also a hard shoulder between the slope and passing traffic. Added to this is the fact that the motorway has a planar, well maintained surface and is situated on a fairly gentle gradient.

Engineering factors: The slope was pre-split blasted and a range of protective and reinforcement measures installed including wire mesh netting, rockbolts and crest and slope drainage. There is evidence that rock material weathering has occurred, probably due to moisture retention, in the relatively protected environment behind many boltheads.

Excavated slope characteristics: On the layered slope there is a significant cover of grass and sporadic shrubs. There are no shrubs in the fissile areas, and less grass cover. The slope has been cut to a height of 11m at a gradient of 64° from the horizontal. Layered parts of the slope have few intersecting fractures and therefore adjustment K2.a must be applied. However, layered parts of the slope are also highly variable, containing very blocky and some intensely fractured zones, as well as small areas of fissile rock within it. A positive adjustment for variability therefore also needs to be incorporated.

Other adjustments: The slope was excavated in 1971 and therefore, at the time of writing had been excavated for 29 years. Excavation was by pre-split blasting, though this has not been completely successful. The slope is extremely unlikely to be disturbed, except for infrequent maintenance operations.

On the basis of the above observations, a total adjustment of +14 was made for layered/blocky slopes, and +9 for fissile slopes calculated as shown in Table 9.8.

The adjusted RDA_A Rating for layered/blocky slopes, therefore, is 28 + 14 = **42**, which is a class 2/3, low to moderate risk slope, requiring a passive to semi-active approach to mitigation. The adjusted RDA_A Rating for fissile slopes is 50 + 9 = **59**, which is a class 3/4, moderate to high risk slope, requiring a semi-active to active intervention approach to mitigation.

LAYERED/BLOCKY SLOPE			FISSILE SLOPE		
Code	**Description or value**	**Rating**	**Code**	**Description or value**	**Rating**
A1.c	High exposure and moderate altitude	3	A1.c	High exposure and moderate altitude	3
B1.a/b	North easterly aspect	2	B1.a/b	North easterly aspect	2
C1.c	Groundwater seepage and surface runoff	2	C1.c	Groundwater seepage and surface runoff	2
G1	Deterioration associated with stabilisation works	3	G1	Deterioration associated with stabilisation works	3
H3.a	Grass cover	3	H3.a	Grass cover	2
K2.a	Regular, non-intersecting structure	-3	L1.d	Time since excavation and pre-split blasting method	-3
K3	Highly variable rock mass	7			
L1.d	Time since excavation and pre-split blasting method	-3			
	Total adjustment	**+14**		**Total adjustment**	**+9**

Table 9.8 Total RDA Rating adjustment for M6 Dillicar

9.3.3.2 Stage Two RDA

Layered/blocky slope: This part of the slope generally fits into the layered type but there are also regular blocky areas within it. The relevant data sheets (Figures 8.11 and 8.14) indicate a typical RDA_A Class for metamorphic rocks of 1 to 3 (layered) and 2/3 (regular blocky). For layered slopes, major deterioration modes can be expected to be stonefall and stone ravelling, with minor modes of wash erosion and blockfall. Grain ravelling, flaking and rockfall might also occur. For regular blocky slopes, stone ravelling is also a major mode, with minor modes of stonefall, blockfall and rockfall. Minor modes include grain ravelling, flaking and wash erosion. Field observation showed that structural chutes were common, often with piles of stones at the foot indicating stone ravelling and small rockfalls (Plate 9.6). The latter were also indicated by several large rockpiles including blocks of 30 to 40cm. A general scatter of isolated stones and blocks at the foot indicated that stonefall and blockfall had also occurred (Plate 9.6). There was also evidence of draped grass on the slope indicating wash erosion (Plate 8.6E).

Plate 9.6 General view of M6 Dillicar showing the range of debris at the slope foot

Fissile slope: This part of the slope neatly fits into the fissile (layered) type. The relevant data sheet (Figure 8.12) indicates a typical RDA_A Class of 1 to 4 for metamorphic rocks, and associated major deterioration mode of flaking, with grain ravelling and wash erosion as minor modes. Stone ravelling, stonefall and rockfall might also occur. There was considerable evidence of waterflow and associated erosion over the surface, indicated by flattened and draped grass, accumulations of fines and soil on ledges, the presence of vegetation, especially a widespread cover of grass. There were also several soil overhangs near the top of the slope and associated debris at the foot indicating grain ravelling. There were steep piles of platey debris at the foot of near vertical fissile layers indicating flaking. These bands were commonly up to 30cm in width and were effectively acting as erosion chutes. In some localities where flaking was undermining more competent strata above, a range of stones and blocks were scattered at the foot, indicating limited stone ravelling, stonefall and blockfall.

9.3.3.3 Stage Three RDA

Assuming that the consequences of deterioration would be unacceptable if no action were taken, the recommended mitigation for this slope would be as follows: *Layered/blocky areas:* Regular debris clearance and removal of woody shrubs; crest drainage and inclined drainholes at critical slope locations; containment using rocktrap ditch and fencing; local wire mesh netting for areas of stonefall and ravelling, with dowel reinforcement of key blocks for areas prone to blockfall. *Fissile areas:* As for layered/blocky areas (since these two zones are part of the same slope), and toe and slope drainage; very close mesh wire or plastic netting for areas with flaking; shotcrete or local mortar screeding for small, weak areas; dentition for small overhangs; and shotcrete sealing of fracture and erosion chutes.

9.4 Practical Application of RDA

The Rockslope Deterioration Assessment method has been designed for use in engineering practice. It is designed to be used for existing and proposed slopes as described earlier. Its use will be most effective if used as a *guide and a tool* for drawing attention to (i) the influences and controls on deterioration; (ii) deterioration risk; (iii) modes of deterioration; and (iv) interpretation of deterioration morphology. As with any new classification scheme or procedure, operatives need to be trained in its use and results obtained from its application need to be reproducible. The classification method needs to be easy to use, the principles easy to understand, and the results straightforward to interpret and apply. The procedure also needs to fit neatly into current project development stages. These issues are addressed in the following sections.

9.4.1 Application of RDA in relation to project development stages

During the *desk study* phase for a proposed excavation, information should be available on the geological formation, including the general nature of lithology present. This, together with any additional information from boreholes, existing excavations and mine records, for instance, should enable an estimate of the likely rock mass type. Consideration of the relevant data sheet will indicate the likely range of RDA Class, deterioration modes and special features to be considered. This information is useful for outline planning of excavation method, siting, phasing of operations (particularly in the context of mineral extraction) slope geometry and post-excavation stabilisation, protective measures and maintenance operations. The resource and financial implications of these can also be estimated and built into the design process. This will be particularly useful for D.B.F.O (Design, Build, Finance and Operate) contracts where the costs of maintenance and ongoing remedial works form part of the initial tender price. It might also prove useful in the design of restoration measures for quarry units, which usually have to be completed in detail prior to obtaining planning permission. Deterioration risk, for example, might have a bearing on the suitability of different after-uses.

Preliminary field investigation will enable some verification of data already available, particularly if exposures exist. It might also be possible to refine information on rock material (eg weathering grade, rock strength) and mass (fracture types, spacing and aperture) properties. Again, this information can be improved and refined during a *detailed site investigation*, but the nature of data obtained will depend to a large extent on the methods used. For instance, borehole sampling will provide greater coverage of variations in properties with depth but will be less useful for refining the model of rock mass structure and its spatial variability. As mentioned above, all of the information obtained can be utilised in the *design process* to minimise the nature of the hazard and the degree of risk, and to better predict financial and resource implications of deterioration. However, prior to excavation, it should be possible to assess the RDA_A Rating and to progress through all three stages of RDA, albeit in a less precise form than will be possible post-excavation.

Immediately *after excavation* (and even as it is progressing), RDA should be undertaken and the results input into the ongoing design process. It is recommended that the face be cleaned, perhaps using a high pressure water jet followed by scaling of loose material. The slope should then be surveyed, and a photographic record made. For investigations made at a later stage, vegetation might also need to be removed as part of the scaling operation in order to be able to undertake proper survey. RDA is a rapid on-site procedure and so it is recommended that assessment of deterioration risk be repeated at all subsequent inspection visits, surveys and *maintenance operations*. During the operation phase, repeated RDA can serve not only to continually update information, but also to highlight new areas of risk, and accumulate information on magnitude and frequency of deterioration events.

9.4.2 RDA training and reproducibility

Over a period of three years, *MSc Engineering Geology* students and undergraduates of Geoscience and Civil Engineering degrees taking a level 3 module in *Rock Engineering* have used an earlier version of the RDA method during fieldwork at Ilkley Quarry, West Yorkshire and Borrowdale cutting on the A685 in Cumbria. The method has been applied by the students with a good level of success without them having been introduced to the method prior to use in the field. The results obtained from these student groups were good considering there had been no training given, and broadly comparable to assessments made by the author at those sites. It is notable that a fellow engineering geology PhD student was able to very closely replicate the authors' own assessment for the two sites mentioned above. Clearly the students' results would have been more polished had they had the opportunity to consider the procedure and assimilate the fundamental basis of the classification prior to use. Nevertheless, this illustrates that a only minimal level of training would be required for it to be used successfully for persons with a training in engineering geology or geomorphology. It should be noted that the students made a number of very valuable comments on the RDA relating to ease of use, explanatory notes and basic principles, and these have been incorporated into the version presented in this thesis.

9.4.3 Field requirements for RDA

RDA can be undertaken quickly and with minimum use of equipment. Experience suggests that the time required to undertake a full RDA depends on the level of pre-existing knowledge of the site, the amount of spatial variability and the skill and experience of the operator. A complex slope which needs to be sub-divided into say, five or six separate zones, might take two hours to fully assess, but a less complex slope could be assessed in around 30-60 minutes, including photography. If a pre-excavation RDA has already been undertaken on the basis of desk study and other preparatory work, a refined RDA could probably be undertaken in 15-30 minutes. In terms of equipment, all that is required is a standard data proforma (Figure 8.3), a geological hammer and Schmidt hammer if available ('N' type recommended for rock exceeding 30MPa), a tape measure, a calliper or ruler for measuring apertures, compass clinometer and an Ordnance Survey map to determine altitude.

In the long term, it is intended that an RDA manual should be published. This would contain all of the data sheets for rock mass types, deterioration modes and deterioration morphology. It would also contain the data collection proforma, ratings charts, adjustment factors tables and associated charts (eg for excavation method and rock mass structure adjustment). The tables indicating general and detailed treatment measures would be included, as would the interaction matrix produced in Chapter Two. However, it is not necessary for all of these components to be used in the field investigation. All that is needed, is a copy of the BS5930 (1999) guide to field estimation of rock strength and the weathering grade classification presented in Chapter Eight (Table 8.1). Where existing rockslopes are being investigated which contain a variety of deterioration forms, it might also be useful to refer to the relevant data sheets for deterioration morphology, but this can just as easily be achieved by making a photographic record of the features concerned, and interpreting them back in the office.

CHAPTER TEN
CONCLUSIONS

10.1 Conclusions

The aim of this research was to investigate the mechanisms and morphology of excavated rockslope deterioration and to design a method for its characterisation and assessment.

10.1.1 Rock deterioration at the material scale

The first objective was to investigate breakdown mechanisms, the role of rock fractures and other material properties on weathering and the mode and severity of resulting deterioration. This was achieved by subjecting rocks to simulated weathering processes, also enabling some evaluation of the effects of varying environmental conditions. The key conclusions of Part One of the thesis are summarised below.

10.1.1.1 Experimental methods

Test conditions influence the results of experimental weathering and this can be seen in the pore infilling which resulted from salt weathering and the abrasive effects of the slake durability test. There were also indications that some specimens which resisted deterioration would have responded had further cycles of weathering been conducted.

Experimental weathering results must be interpreted in relation to the environmental conditions to which samples were subject because some samples will deteriorate in a completely different way for different simulated weathering processes. In other cases, the severity of deterioration differed for each weathering test in samples which deteriorated by the same mode.

It is useful to use a range of deterioration indicators in experimental weathering tests, particularly if each relates to a different facet of rock breakdown. In this case, weight loss measured material detachment, fracture density measured in situ fracturing, and sonic velocity was used to measure internal changes in void volume. Other measures of mineral alteration and change in elasticity could also be used.

10.1.1.2 Influences and controls on rock deterioration

For most of the samples tested, rock properties exerted a dominant control on deterioration response, with environmental control being subservient. Rock properties which exerted greatest control on deterioration susceptibility were increasing total pore volume, saturation coefficient and microporosity. There was also a good correlation between strength or elasticity and durability for the stronger rocks. For moderately strong or weak rocks, however, the correlation was much less clear. Some of these properties are relatively easy to estimate in the field and therefore approximate predictions of behaviour can be made on this basis.

For a few rocks environmental conditions exerted greater influence on deterioration response. In some cases the mode of deterioration was unchanging with different weathering tests, but severity of breakdown did change. This partly reflects the fact that some weathering conditions (eg freeze-thaw) were generally more severe than others (eg wetting and drying). In other rocks, different weathering conditions produced completely different styles of breakdown. This partly reflects that fact that some weathering conditions are associated with given modes of deterioration (eg freeze-thaw leads to fragment detachment, slaking leads to attrition and salt weathering leads to in situ fracturing).

In the context of field evaluation of deterioration, it is likely that minor changes in environmental conditions or rock properties could give rise to contrasting mode and/or severity of deterioration in essentially the same rock.

It is not possible on the basis of the research conducted here, to identify a single rock property which would enable accurate prediction of weathering behaviour. Relations between rock deterioration, material properties and environmental conditions are extremely complex. Although general trends can be identified it is uncertain whether it will ever be possible to accurately predict deterioration response (severity and mode) of a specific area of a rockslope. The weathering mechanisms are still not understood fully, nor the rock and environmental controls involved, and even some rock properties are difficult to measure accurately. Nevertheless, some appreciation of general trends and the range of properties and conditions likely to exert an influence can enable a general assessment of likely behaviour under given conditions, though any such assessment must be treated with caution.

10.1.1.3 Rock deterioration mode and the role of rock flaws

Only some rock deterioration is visible in hand specimen. Internal modification of voids was detected by sonic velocity measurements and also by monitoring changes in pore properties. This must be borne in mind when assessing rockslopes in the absence of similar test data.

A range of rock flaws have been identified and described. They are a major influence on the severity and mode of deterioration. Linear flaws, in particular, are more likely to exert a major influence on breakdown and therefore need to be recorded in field investigations in addition to major joint sets. The penetration and persistence of flaws is particularly important in determining their influence on deterioration.

Contrary to expectations, stronger rocks were more susceptible to the influence of rock flaws, where present, than weak rocks. The latter deteriorated more in response to mechanical weakness, void properties and texture.

Several distinct modes of breakdown were identified, tending to be closely related to particular rock properties or flaws. Others such as scaling were more dependent upon weathering conditions both in the laboratory and under field conditions.

For some rocks there was considerable similarity between deterioration mode due to experimental weathering and breakdown on the rockslopes from which they were obtained. This shows that (i) laboratory tests were, to some extent, re-producing field conditions, and (ii) breakdown at the material scale is extremely important in deterioration of some rockslopes.

10.1.1.4 Rock breakdown mechanisms

Several mechanisms of internal modification of rocks were identified, including the flushing out or re-distribution of debris from the break-up of grain contacts; microcracking; pore enlargement due to dissolution or wall compression; case hardening; pore infilling; and pore collapse. Together, these result in modifications to existing pores and/or the introduction of new void.

On the basis that macrofractures are generated from modifications to microcracks and pores, a succession of three progressive breakdown stages was proposed. Initially, internal modifications to voids occurred which do not result in any change to total pore volume and do not produce weakening. The second stage involves an increase in total pore volume due to modification of existing pores. In the final stage, new void is introduced into the rock, accompanied by significant weakening.

For all but the strongest rocks, weathering resulted in a reduction in rock strength and elasticity, even in the absence of any visible evidence of deterioration or surface rupture.

10.1.2 Rockslope deterioration at the mass scale

The second objective was to determine from field investigation of excavated rockslopes the nature and morphology of deterioration and the consequences arising from it. The field investigation also enabled some assessment of the intrinsic and external factors influencing and controlling rockslope deterioration. The key conclusions of Part Two of the thesis are summarised below.

10.1.2.1 The occurrence of deterioration

Deterioration was in evidence for most excavated rockslopes investigated, though the severity, mode, consequences and mitigation of deterioration varied considerably. Excavated rockslope deterioration is largely dealt with on an ad hoc basis, responding to problems as need arises. There is no systematic approach used in the evaluation of rockslopes and there was little evidence of any monitoring of slopes. Under-reaction was more common than over-design. This was largely due to the lack of resources available because deterioration problems had not been anticipated early on.

10.1.2.2 Influences and controls on rockslope deterioration

As expected, reducing strength and fracture spacing correlated well with increasing deterioration. But, smaller, non-persistent fractures arising from stress release, blasting and weathering were much more commonly associated with block release than major discontinuities.

Another major influence on deterioration was the presence of surface water and groundwater seepage, particularly in the context of material-related deterioration. Vegetation, particularly the roots of woody shrubs and trees, was extremely commonly associated with areas of fragmented rock mass, although the precise cause and effect relationship here is unclear.

10.1.2.3 Deterioration morphology

A range of erosional and depositional landforms and process indicators were identified and described. These include chutes, overhangs, cavities, macro landforms, scars, debris piles, scattered and isolated debris, fracture infilling, in situ disintegration and decomposition, vegetation and evidence of water flow.

10.1.2.4 Deterioration modes

Different modes of deterioration could be identified on the basis of morphological forms and process indicators. Modes are classified into semi-continuous, sporadic and isolated types, categorised on the basis of the velocity of movement, constituent material size and frequency. Event magnitude can also be inferred from the deterioration mode.

Deterioration mode closely relates to different rock mass and material properties. There is a notable distinction between deterioration modes influenced by mass properties (eg stonefall and block ravelling) and those influenced more by material properties (eg wash erosion and grain ravelling).

10.1.2.5 Rock mass types

Deterioration mode was closely associated with rock type, fracture network and rock strength. On the basis of these parameters, seven rock mass types were identified. Not surprisingly, there is a good correlation between certain deterioration modes and rock mass types. Other rock masses have a wide variety of deterioration modes associated with them, and other deterioration modes such as scaling and stonefall are ubiquitous.

10.1.3 Rockslope Deterioration Assessment (RDA)

The final objective was to utilise the results of the research to develop a new rock mass classification which addresses the problem of excavated rockslope deterioration. The three-stage Rockslope deterioration Assessment (RDA) method was presented. In stage one, ratings are applied to a range of intrinsic rock properties and external factors to provide a relative measure of risk. In stage two, the nature of potential deterioration is explored using the classifications of deterioration morphology, deterioration mode and rock mass type. In the final stage, guidance is provided on mitigation measures appropriate to the nature and level of deterioration risk.

Application of RDA to over 200 slope units in the UK shows that some rock mass types (eg fissile and composite) are associated with high deterioration risk, while others (eg strong massive) are associated with low risk. Certain types of deterioration mode are more likely in high RDA rated slopes, usually those involving high magnitude (eg debris flow and rockfall) or high frequency (eg stone ravelling). Some deterioration modes occur in rockslopes with a narrow range of RDA ratings while others such as scaling and stonefall occur in a much wider range of rockslopes. A strong trend was evident in which a single, major deterioration mode or only minor modes were likely to prevail on low risk slopes whereas several major modes were likely to co-exist in higher risk slopes.

A reasonable correlation was established between RDA_U and Rock Mass Rating (Bieniawski 1979) for sedimentary and metamorphic rockslopes, with the primary difference between them being the basis upon which fracture spacing is determined. While the RMR considers major discontinuity sets, RDA includes all fractures regardless of origin, including those produced by anthropogenic means, weathering and stress release. For surface and shallow deterioration processes these fractures are of considerable importance.

If used correctly and as intended, as a design and planning tool, RDA has significant potential for use in engineering practice. It is a procedure which is easy to apply, requiring minimum field equipment and training. It is also easy to interpret given an engineering geology or geomorphology training.

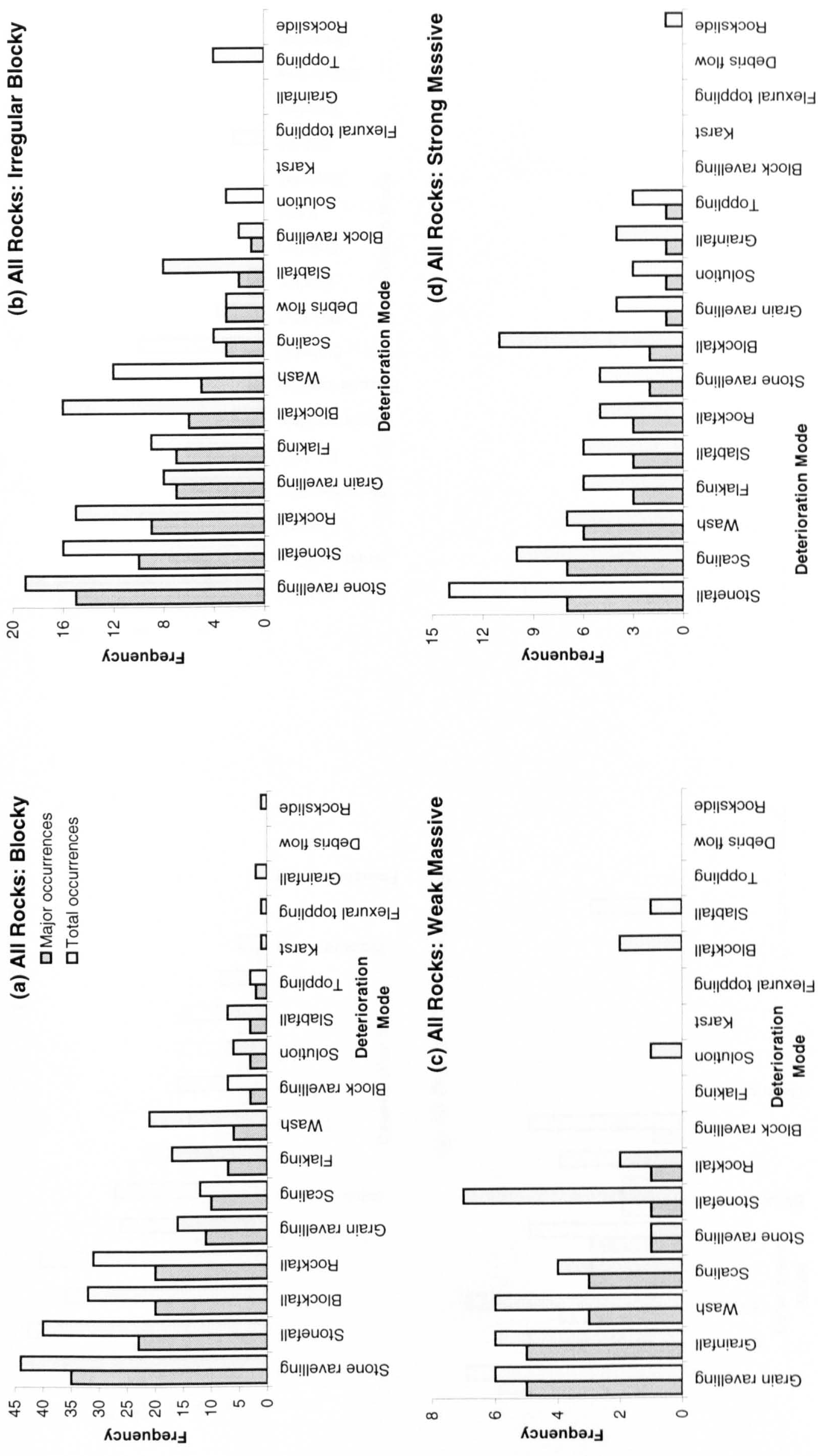
10.2 Recommendations for Further Work

The general methodology applied for experimental weathering has proved useful. However, for future work with similar objectives, it would be useful to make some modifications. The author would recommend use of a balance with greater accuracy than was available for this work. Some further investigation is also needed to account for the intra-sample temporal fluctuations in sonic velocity measurements. Fracture density is an extremely useful and easy to apply deterioration indicator. However, it would be beneficial to determine if its use is comparable at a range of scales and for different specimen shapes. The use of a non-standard method for simulating freeze-thaw means that results cannot be compared directly with the work of others. However, since one of the primary objectives of this research was to induce deterioration in the rocks tested, this is not critical. In the context of studies designed to investigate the mechanisms of freeze-thaw and the specific role of different moisture and temperature regimes, it is *essential* that a variety of test methods are utilised in order to isolate cause and effect relationships and in this respect, the use of 'standards' should be resisted.

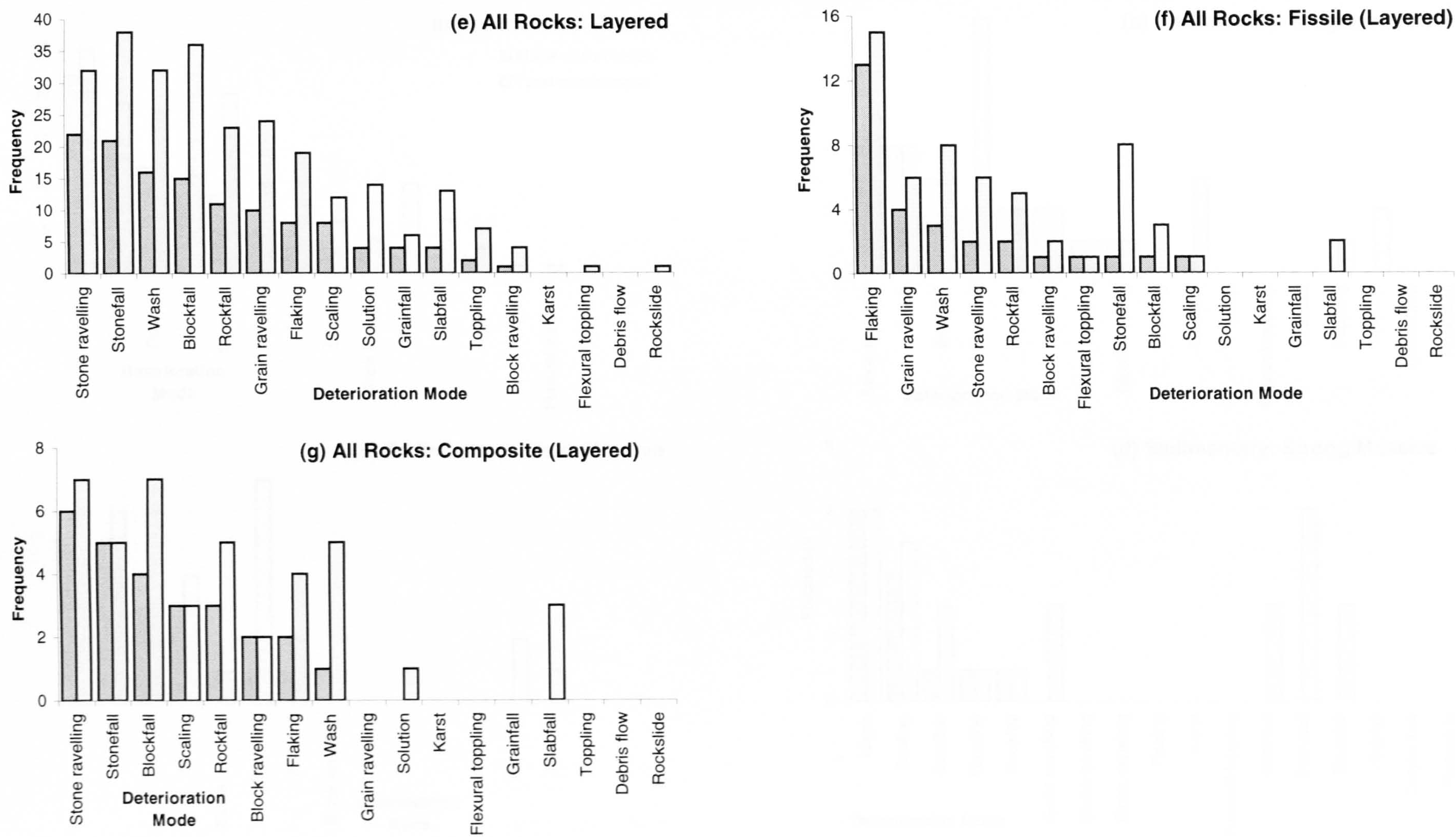
It would be useful to see how much effect increasing the number of weathering cycles has on deterioration of the more durable rocks. It would also be interesting to apply similar testing methods to a wider range of rock types. However, experimental weathering of a much narrower range of rocks, for instance, a range of different density chalks or differentially weathered gritstones, might actually prove more revealing in terms of the mechanisms operating since variations could not readily be attributed to major contrasts in rock structure, texture and mineralogy. The measurement of pre-and post-test void properties using mercury porosimetry has shown to provide considerable insight into modifications of pore size distribution and associated breakdown mechanisms. This work would benefit from application to a much larger number of specimens at each test stage to reduce the potential effects of inter-specimen variations. However, this would be expensive. An alternative would be to investigate the use of air porosimetry which is non-destructive and therefore repeatable.

As with any new classification, it would be beneficial for RDA to be applied to a larger number of excavated rockslopes, particularly in the rock groups not as well represented here (eg igneous and metamorphic slopes, and those which achieve very high and very low RDA ratings). This would enable minor modifications to be made to ratings and adjustments. More significant changes could be made to the adjustments for environmental factors to enable its application in other climatic regions. However, the RDA has not, as yet, been applied and tested for proposed slopes, or new slopes whose deterioration behaviour has subsequently been monitored. This could be a major aim of future work.

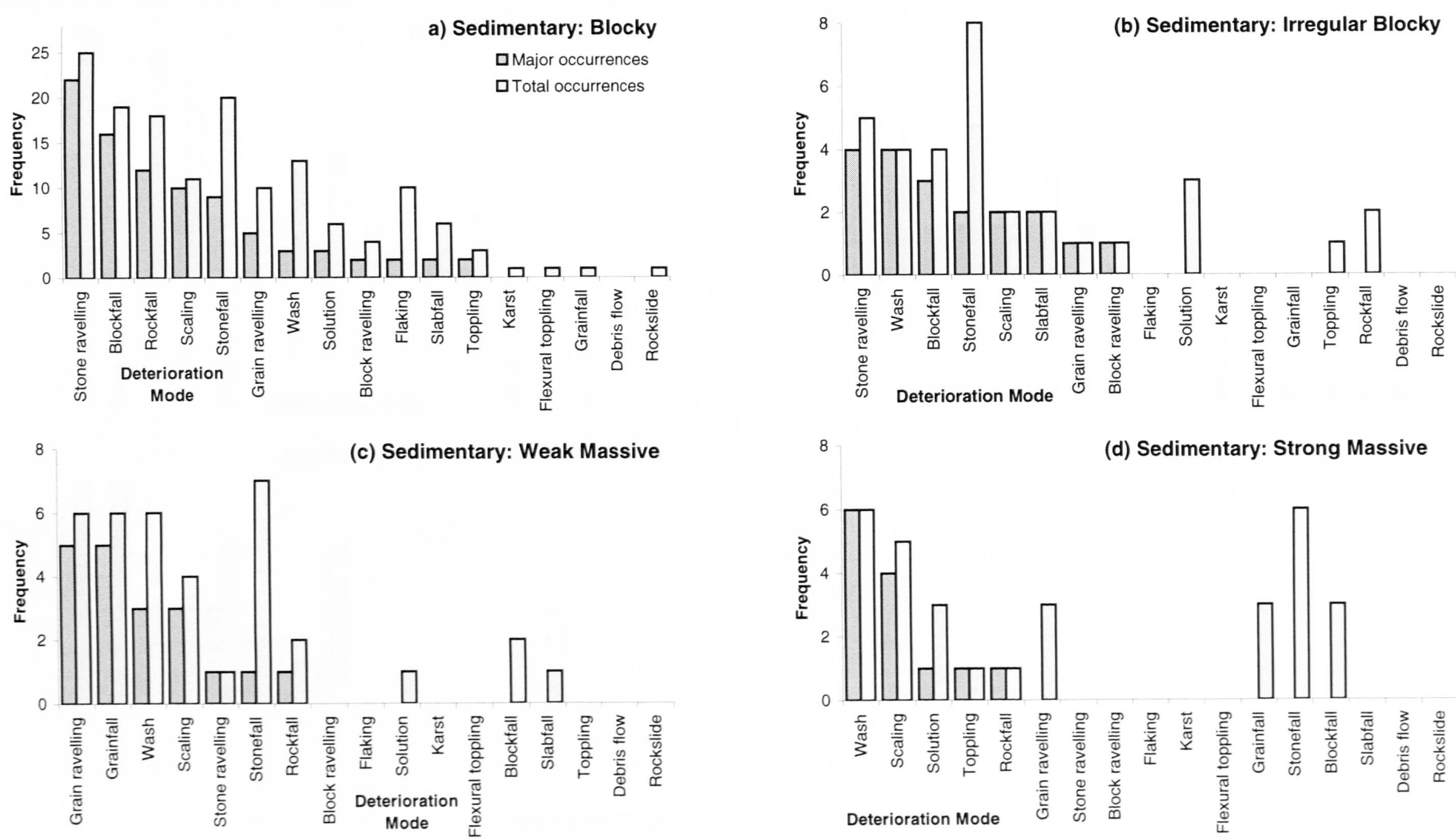
APPENDIX 7.A
ABSOLUTE FREQUENCY DISTRIBUTION OF DETERIORATION MODES FOR EACH ROCK MASS



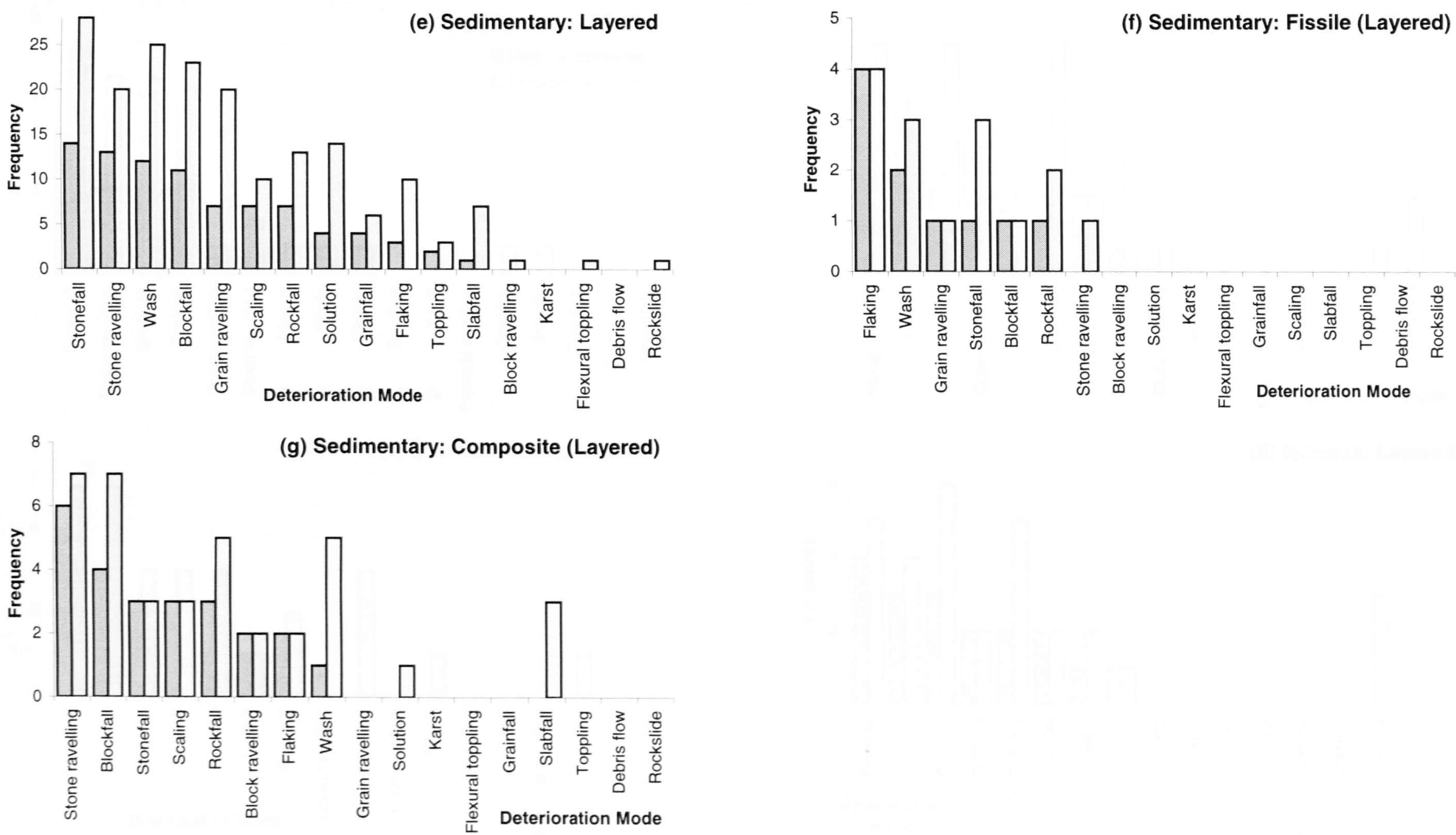
Appendix 7.Aa (a to d) Absolute frequency distribution of deterioration modes for each rock mass type



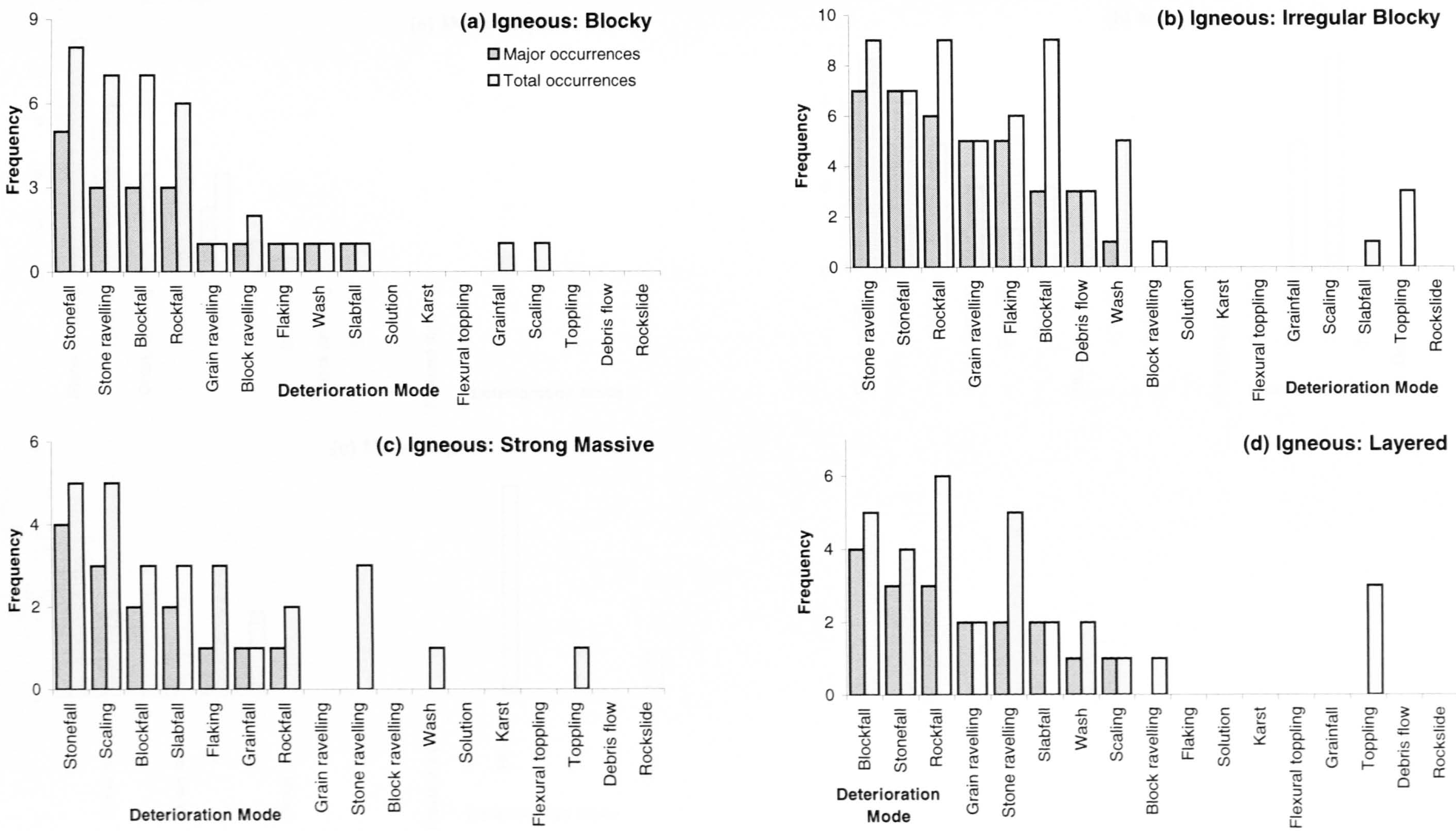
Appendix 7.Aa (e to g) Absolute frequency distribution of deterioration modes for each rock mass type



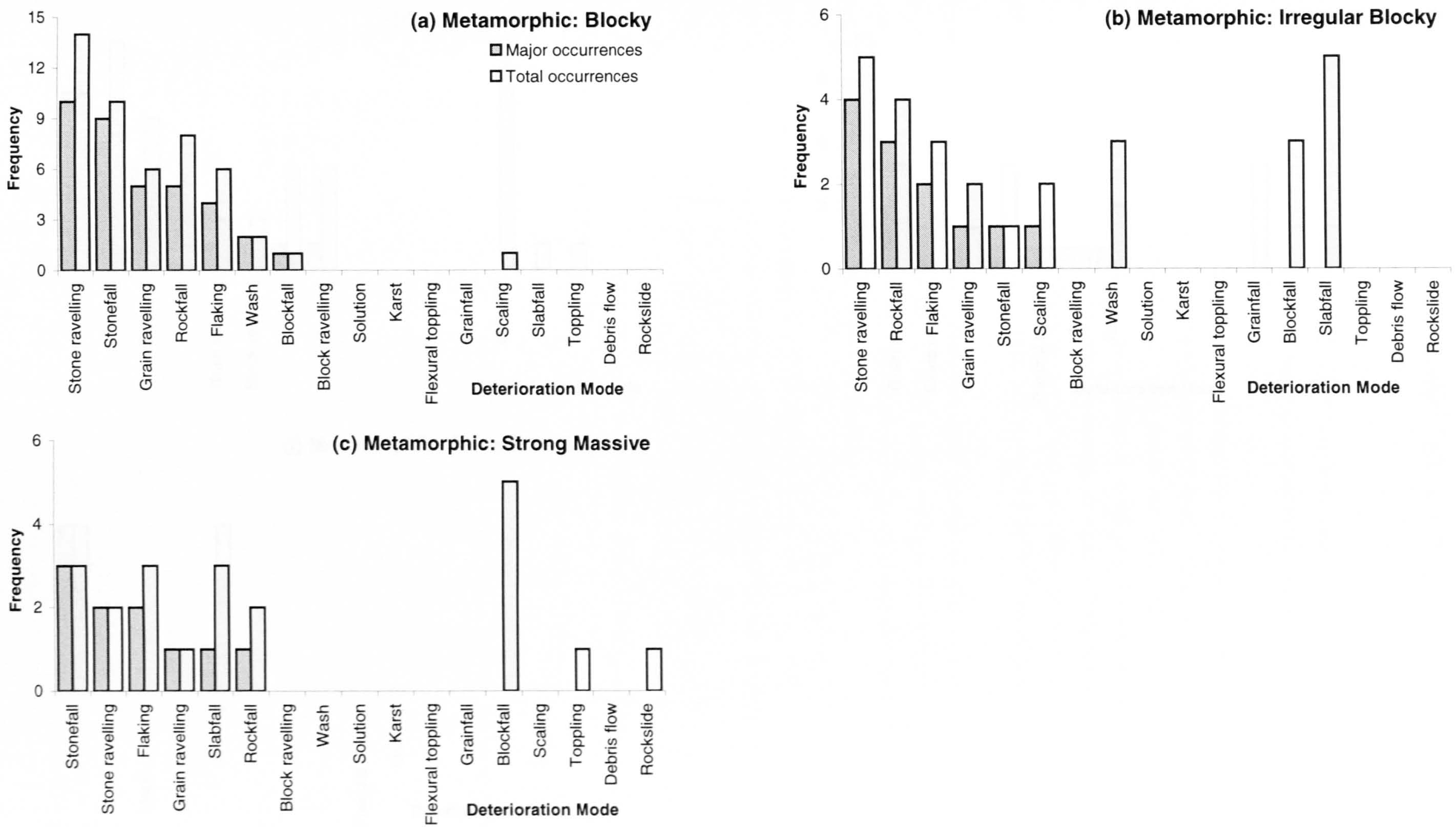
Appendix 7.Ab (a to d) Absolute frequency distribution of deterioration modes for sedimentary rock masses



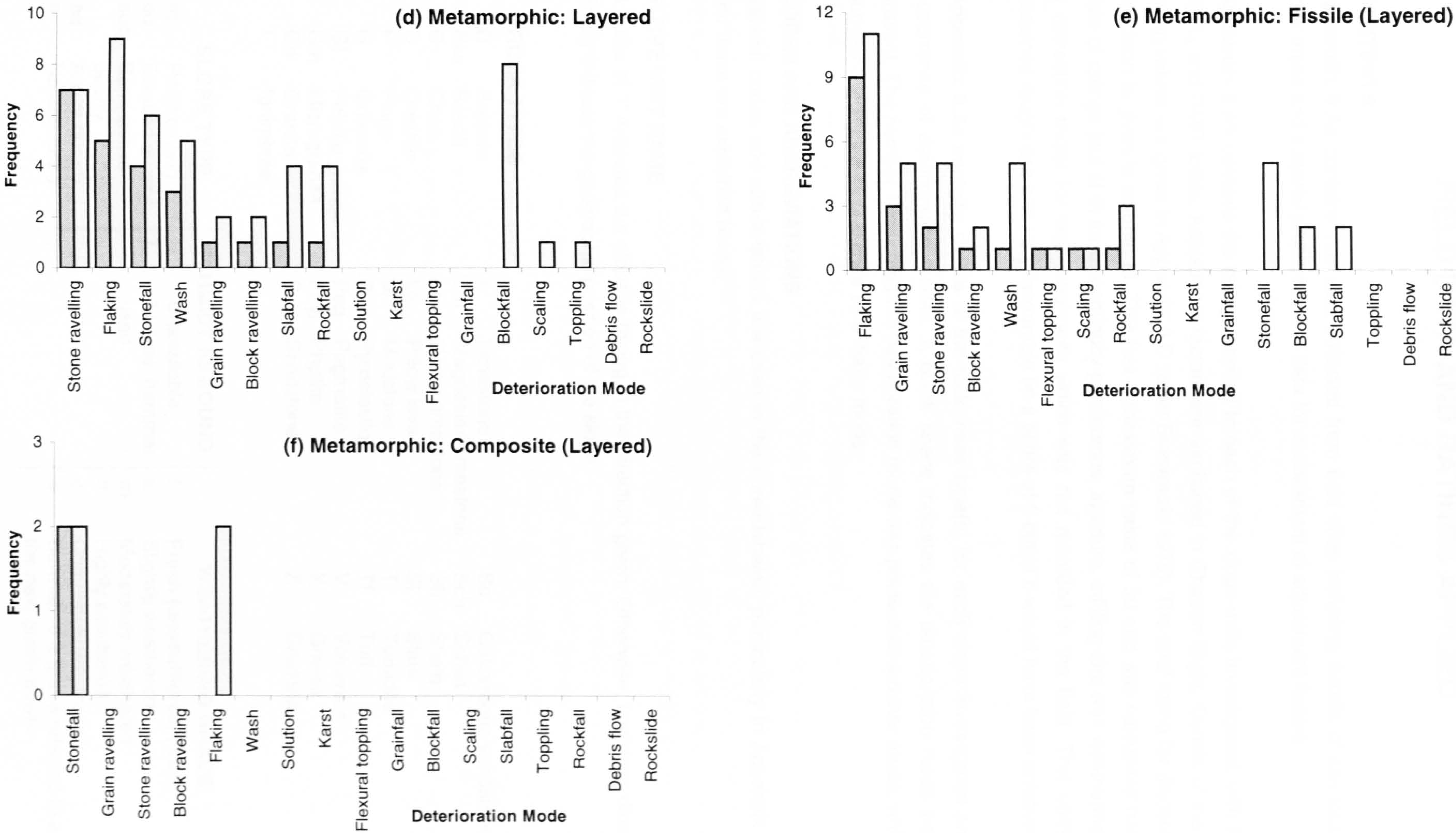
Appendix 7.Ab (e to g) Absolute frequency distribution of deterioration modes for sedimentary rock masses



Appendix 7.Ac (a to d) Absolute frequency distribution of deterioration modes for igneous rock masses



Appendix 7.Ad (a to c) Absolute frequency distribution of deterioration modes for metamorphic rock masses



Appendix 7.Ad (d to f) Absolute frequency distribution of deterioration modes for metamorphic rock masses

APPENDIX 9.A
FIELD DATA AND RATINGS APPLIED

CONTENTS

Appendix 9.Aa contains raw data collected from field sites including details of site locations, rock mass and material properties and data for assessment of adjustment factors.

Appendix 9.Ab contains the ratings applied to each of the slope units investigated, with RDA_U, RDA_A and RMR totals. Adjustment factors are explained in Chapter Eight. Details of the RMR rating values are given in Appendix 9.C (*after Bieniawski 1979*). The total rating for discontinuity condition is given in column M. This has a maximum value of 30 and was calculated from the sum of ratings (out of 6) for discontinuity persistence, aperture, infilling and wall weathering, plus a correction factor for wall roughness which was not recorded in the field. This correction assumes roughness was very favourable (ie a score of 6 out of 6 would have been achieved).

Appendix 9.Ac contains details of the rock mass type(s) for each slope investigated and the occurrence of deterioration modes. A blank space indicates the deterioration mode was not present. The number 1 indicates the deterioration mode was present as a minor mode, while the number 2 indicates its presence as a major mode.

CODES AND ABBREVIATIONS

Several codes and abbreviations are used in the spreadsheets, particularly in Appendix 9.Aa, and these are explained below.

SLOPE UNIT NAME

A suffix of 'f' indicates the slope is <u>facing</u> in the direction given. Otherwise, compass directions simply indicate the geographic position of the slope.

ROCK TYPES

B	Breccia	L	Limestone	Sc	Calcareous sandstone
Bas	Basalt	Lm	Magnesian limestone	Sch	Schist
C	Chalk	Lo	Oolitic limestone	Sh	Shale
D	Dolerite	Lp	Pillow lava	Sl	Slate
F	Flags	M	Mudstone	T	Turbidite
G	Gritstone	P	Pyroclastics	Tf	Tuff
Gf	Felsitic granite	Peg	Pegmatite	V	Volcanics
Gm	Microgranite	Phyl	Phyllite	X	Gneiss
Gn	Granite	S	Sandstone	Z	Siltstone
I	Ignimbrite				

SLOPE TYPE

rc	Road cutting
dq	Disused quarry
saq	Semi-active quarry
aq	Active quarry
n	Natural slope

SCHMIDT REBOUND

u	Unavailable
L	'L' type hammer used

WEATHERING GRADE

f	Fresh (unweathered, RMR)
s	Slightly weathered
m	Moderately weathered
h	Highly weathered
c	Completely weathered
+	Between the grade referred to and the next grade down

PERSISTENCE (RMR)		FRACTURE INFILL (RMR)		FAVOURABILITY (RMR)		EXPOSURE LEVEL	
1	<1m	c	Clean (none)	1	Very favourable	1	Sheltered
2	1-3m	ha	Hard <5mm	2	Favourable	2	Slightly exposed
3	3-10m	hb	Hard >5mm	3	Fair	3	Moderately exposed
4	10-20m	sa	Soft <5mm	4	Unfavourable		
5	>20m	sb	Soft >5mm	5	Very unfavourable	4	Very exposed

GROUNDWATER FLOW

Categories correspond to the five classes used by both the RMR and RDA (refer to adjustment C). 1 relates to low flow or dry conditions and 5 relates to the wettest conditions. 'e' indicates *evidence* of groundwater flow rather than actual flow.

EXCAVATION METHOD

b	Bulk blasted
h	Hand excavated
m	Mechanically excavated
n	Natural slope
pb	Pre-split blasted
u	Unknown

RDA FAVOURABILITY

f	Favourable
u	Unfavourable
v	Very unfavourable

TIME SINCE EXPOSED

u	Unknown

ROCK MASS TYPE

B	Blocky	Lc	Composite (layered)	Pw	Pillow structure
Bi	Irregular blocky	Lf	Fissile (layered)	R	Rubbly (chalk, oolitic limestone)
Bp	Blocky prismatic	Ms	Strong massive		
C	Composite	Mw	Weak massive	V	Vertical layering
L	Layered			Y	Yes, if intensely fractured zones present (IFZ)

GENERAL NOTES ON HOW TO READ THE SPREADSHEETS

For Appendices 9.Aa and 9.Ab pages read first across and then down. In both cases, the spreadsheet is two pages wide and three pages deep. Appendix 9.Ac is a single page wide and three pages deep. Where appropriate, the site number, given as column **A**, is repeated at the left hand edge of each sheet. Column headings are not re-produced in full, but a letter in bold, corresponding to each column heading, is re-produced for each page down the spreadsheet.

In Appendix 9.Ab some ratings are prefixed with 'x' (notably in columns AB to AH pertaining to adjustment H). These are not included in the Total Adjustment because more than one adjustment was relevant for similarly numbered items in a sub-section. For instance, in sub-section H relating to vegetation cover on a slope, adjustments H.3a, H.3b and H.3c might all apply, indicating that grass, shrubs and trees were present on a particular slope. In such a case, only the single highest rated item is incorporated into the Total Adjustment and others are prefixed with 'x' to show they apply.

Site number	Slope unit number	Site name	Grid reference	Slope unit name	Rock type	Slope type	Fracture spacing (cm)	Fracture aperture (mm)	Rock strength (MPa)
A	B	C	D	E	F	G	H	I	J
1	1	Aberford cutting, A1/M1 link roac	SE443436		Lm	rc	6	0.4	8
2	2a	Addingham Moorside	SE095470	Upper	G	dq	60	3	60
3	2b	Addingham Moorside	SE095470	Lower	G	dq	150	2	60
4	3a	Bankend Quarry, St Bee:	NX993127	1 upper	S	dq	95	0.5	40
5	3b	Bankend Quarry, St Bee:	NX993127	1 lower	S	dq	80	2.5	40
6	3c	Bankend Quarry, St Bee:	NX993127	2	S	dq	60	0.5	40
7	3d	Bankend Quarry, St Bee:	NX993127	3	S	dq	80	2.5	40
8	3e	Bankend Quarry, St Bee:	NX993127	4	S	dq	95	0.5	40
9	3f	Bankend Quarry, St Bee:	NX993127	5	S	dq	60	0.5	40
10	3g	Bankend Quarry, St Bee:	NX993127	6	S	dq	60	0.5	40
11	4a	Banks Gate, A66 Brough	NY843148	Old upper	L	rc	30	2	80
12	4b	Banks Gate, A66 Brough	NY843148	Old lower	L	rc	40	0.5	80
13	4c	Banks Gate, A66 Brough	NY843148	New	L	rc	15	2	80
14	5a	Beamsley A5S	SE087533	Upper	G,S,Sh	rc	31	7	50
15	5b	Beamsley A5S	SE087533	Lower	G,S,Sh	rc	113	4	50
16	6	Belah Scar, Eden Valley (RIGS	NY794121		S,B	n	250	0.2	4
17	7a	Bidston Hill, Wirral	SJ287895	N 1	S	rc	150	0.5	20
18	7b	Bidston Hill, Wirral	SJ287895	N 2	S	rc	20	0.5	40
19	7c	Bidston Hill, Wirral	SJ287895	S 1	S	rc	150	0.5	20
20	7d	Bidston Hill, Wirral	SJ287895	S 2	S	rc	20	0.5	40
21	7e	Bidston Hill, Wirral	SJ287895	S 3	S	rc	3	0.5	20
22	8	Birkhams Quarry, St Bee:	NX955154		S	saq	180	0.5	42
23	9	Birkrigg Crossroads	SD281745		L	dq	10	2.5	100
24	10	Birkrigg Commor	SD283740		L	dq	80	0	120
25	11	Blubberhouses A5S	SE137552		G	rc	100	1	14
26	12	Bongate Scar, Eden Valley (RIGS	NY687199		S	dq	250	2	4
27	13a	Brathay Quarries, CGS No. 3	NY357016	1	M,Z	saq	220	10	90
28	13b	Brathay Quarries, CGS No. 3	NY357016	2	M,Z	saq	100	4	90
29	13c	Brathay Quarries, CGS No. 3	NY357016	Sf 3	M,Z	saq	55	2	90
30	13d	Brathay Quarries, CGS No. 3	NY357016	Nf 3	M,Z	saq	55	2	90
31	13e	Brathay Quarries, CGS No. 3	NY357016	Ef 3	M,Z	saq	55	2	90
32	14a	Church Lane, Church Brougl	NY793193	1	S	rc	190	5	20
33	14b	Church Lane, Church Brougl	NY793193	2	S	rc	190	5	20
34	15a	Clitheroe A59, Lancashire	SD774445	1	L,Sh	rc	12	2	100
35	15b	Clitheroe A59, Lancashire	SD774445	2	L,Sh	rc	20	1.5	100
36	16	East Chevin Quarries	SE213445		G	dq	150	0.2	40
37	17a	Elland Road Bypass, Halifa:	SE118204	High	S,F	n?	100	5	40
38	17b	Elland Road Bypass, Halifa:	SE118204	Low	Sh	n?	2	1	4
39	17c	Elland Road Bypass, Halifa:	SE103216	A629	S	rc	70	1.5	45
40	18	Faulds Brow Quarry, Caldbecl	NY303407		L	dq	40	0.2	80
41	19a	Frizington Park Quarry	NY039156	Bedding	L	dq	75	0.2	90
42	19b	Frizington Park Quarry	NY039156	Main	L	dq	33	3	90
43	19c	Frizington Park Quarry	NY039156	Fractured	L	dq	10	5	90
44	20a	Glenarm, A2, N. Irelanc	D331139	New	C	dq,rc	8	0.2	10
45	20b	Glenarm, A2, N. Irelanc	D331139	Old	C	dq	6	0.2	10
46	21	Godley Cutting, A58(T), Halifa:	SE101257		Sh,M	rc	0.5	0.5	4
47	22a	Grimston, Malton, North Yorkshire	SE847674	Fractured	Lo	dq	7	2	50
48	22b	Grimston, Malton, North Yorkshire	SE847674	N end	Lo	dq	100	1	50
49	22c	Grimston, Malton, North Yorkshire	SE847674	Blocky	Lo	dq	20	2	50
50	23	Harden Moor Quarry	SE972384		G	saq	30	2	60
51	24a	Harmby Quarry	SE125903	Upper	L,Sh,S	dq	12	1	30
52	24b	Harmby Quarry	SE125903	Lower	L	dq	100	0.4	80
53	25	Helbeck Quarry	NY799157		L	dq	20	3	50
54	26a	High Cross Plantation Quarry	NY328985	Brathay	M,Z	dq	20	1	60
55	26b	High Cross Plantation Quarry	NY328985	Birk Rigg	S,T	dq	40	0.5	80
56	27	Hoff Quarry	NY676180		B	dq	250	14	120
57	28a	Hovingham Quarry	SE675750	Blocky	Lo	dq	10	4	70
58	28b	Hovingham Quarry	SE675750	Massive	Lo	dq	50	7	70
59	28c	Hovingham Quarry	SE675750	Layerec	Lo	dq	25	3	70
60	29	Ilkley Quarry, West Yorkshire	SE128467		G	dq	100	2	60
61	30a	Kepwick Quarry, North Yorkshire	SE486914	Nf	Lo	dq	12	4	50
62	30b	Kepwick Quarry, North Yorkshire	SE486914	Wf	Lo	dq	12	4	50

APPENDIX 9.Aa Field site data values

Site number	Schmidt (N) hammer rebound	Weathering grade	RQD (%)	RMR discontinuity spacing (cm)	Discontinuity persistence	Fracture infilling	RMR favourability	Altitude (m)	Coastal location	Exposure level	Aspect	Groundwater flow	Excavation method	Stabilisation measures	Slope height (m)	Slope gradient (deg)	RDA favourability	Time since exposure
A	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB
1	u	m	0	6	3	sa	1	55	n	2	W	1	m	n	6	63	f	0.5
2	u	s	100	60	5	sa	1	270	n	4	N	e2	h	n	6	90	f	150
3	u	s	100	150	5	sa	1	270	n	4	N	e2	h	n	6	90	f	150
4	u	s	100	95	5	sa	1	135	y	2-3	E	e2	h	n	9	86	f	u
5	u	s	100	95	4	sa	1	135	y	2-3	E	e2	b	n	9	86	f	u
6	u	s	90	95	5	sa	1	135	y	2-3	E	e2	h	n	9	86	f	u
7	u	s	100	95	5	sa	1	135	y	2-3	S	e1	b	n	16	86	f	u
8	u	s	100	95	5	sa	1	135	y	2-3	E	e2	h	n	16	86	f	u
9	u	s	100	95	5	sa	1	135	y	2-3	W	e2	h	n	5	86	f	u
10	u	s	100	95	5	sa	1	135	y	2-3	S	e2	h	n	5	86	f	u
11	u	f	0	30	5	sb	1	325	n	4	SW	e3	b	n	4	85	f	>50
12	u	f	100	40	5	c,ha,sa	1	325	n	4	SW	e3	b	n	4	85	f	>51
13	u	f	50	70	5	c,ha,sa	1	325	n	4	SW	e3	b	n	6	75	f	2
14	u	m	80	50	5	c,sa	1	200	n	1	SE	e1	b?	y	10	76-83	f	u
15	u	m	100	150	5	c,sa	1	200	n	1	SE	e1	b?	y	10	76-83	f	u
16	u	m	0	250	5	c	1	160	n	1	NW	e3	n	n	10	85-110	f	n
17	u	m	100	150	4	sa	1	70	n	1	NW-N	3	b,h	n	8	80	f	>80
18	u	m	50	30	3	sa	1	70	n	1	NW-N	3	b,h	n	8	80	f	>80
19	u	m	100	150	4	sa	1	70	n	1	SE-S	3	b,h	n	8	80	f	>80
20	u	m	50	30	3	sa	1	70	n	1	SE-S	3	b,h	n	8	80	f	>80
21	u	m+	0	3	2	sa	1	70	n	1	SE-S	3	b,h	n	8	80	f	>80
22	53	s	100	250	5	sa	1	100	y	2	all	e1	b,h	n	23	90-95	f	19
23	57	f	20	14	4	sa,sb	2	90	n	4	W	e1	b	n	8	60-82	f	30-50?
24	59	f	100	80	3	ha	1	120	n	4	SW	e2	?	n	3	90	f	>80
25	u	m+	100	100	4	ha, sa	2	300	n	3	N	3	b	n	8	85	f	?20
26	u	s	0	250	5	sa	1	145	n	1	W	e2	h	n	15	86	f	>80?
27	69	f	100	220	5	c	1	80	n	2	W	e1	b	n	4-20	u	f	<10?
28	69	f	100	100	5	c	1	80	n	2	E-SE	e2	b	n	4-20	u	f	<10?
29	69	f	100	55	5	c	4	80	n	2	S	e1	b	n	4-20	u	u	<10?
30	69	f	100	55	5	c	1	80	n	2	N	2	b	n	4-20	u	f	<10?
31	69	f	100	55	5	c	1	80	n	2	E	2	b	n	4-20	u	f	<10?
32	u	s+	100	190	4	sa	1	160	n	1	W	2,e3	h	n	4	85-100	f	>100
33	u	s+	100	190	4	sa	1	160	n	1	E	2,e3	h	n	4	85-100	f	>100
34	59-64	f+	50	25	3	ha,sa,sb	1	90	n	2	W	4	b	n	14	83	f	<30?
35	59-64	f+	100	90	4	ha,sa,sb	1	90	n	2	W	4	b	n	14	83	f	<30?
36	u	s	100	150	5	sa	1	210	n	3	N	e2	h?	n	5-18	90	f	>100
37	u	s+	80	100	5	c	1	170	n	3	SW	e1	n?	n	16	85	f	n
38	u	h+	0	0	1	c,sa	1	160	n	3	SW	e2	n?	n	7	65	f	n
39	u	f+	80	70	3	c,ha	1	100	n	2	SE	1	s	n	18	85	f	<30
40	59	f+	90	40	5	c,sa	1	320	n	3	E	e2	b	n	8	80	f	>30?
41	u	f+	100	75	5	c	1	110	n	3	E	3	b	n	10	90	f	>50?
42	u	f+	90	35	4	c,sa	2	110	n	2	SE	2	b	n	6	90	f	u
43	u	s	20	0	1	sa,sb	1	110	n	2	SE	2	b	n	6	90	f	u
44	u	f+	0	30	3	sa	1	10	y	4	NE	1	b?	n	20	80-90	f	<20
45	u	f	0	50	3	sa	3	10	y	4	NE	1	b	n	18	80-90	u	>35
46	u	h+	0	0	1	c	1	190	n	2	NW	e3	h	n	10	80-85	f	100
47	50-54	s+	0	15	4	ha	1	105	n	1	E	1	b?	n	6	85	f	40?
48	50-54	s	100	80	5	ha	1	105	n	1	E	1	b?	n	6	85	f	40?
49	50-54	s	80	80	5	ha	1	105	n	1	E	1	b?	n	6	85	f	40?
50	u	f+	100	30	3	ha	1	290	n	2	All	e1	b	n	10	85	f	5
51	u	s+	30	12	4	sa	1	190	n	2	W-SW	e2	b	n	5	75-90	f	60
52	u	s	100	100	4	sa	3	190	n	2	W-SW	e2	b	n	10	100	u	60
53	u	f+	30	20	3	c,sa	4	300	n	3	SE	e2	b	n	10	90	u	?20
54	66	f	80	60	4	c,ha,sa	1	190	n	2	SW	3	b?	n	9	82	f	>30
55	65	f+	100	60	5	c,sa	1	190	n	2	SW	3	b?	n	9	82	f	>30
56	u	f	100	250	5	c,ha	1	145	n	2	S	e2	b	n	12	77-92	f	>90
57	56	m	0	10	3	sa,sb	1	50	n	2	N	e2	u	n	8	75	f	<20
58	55-60	s	80	80	3	sa,sb	1	50	n	2	N	e2	u	n	8	88	f	<20
59	56	s+	50	25	4	sa,sb	1	50	n	2	N	e2	u	n	8	88	f	<20
60	u	m+	100	100	5	sa	1	250	n	3	W	e1	b	n	8	85	f	>100
61	47	s	50	15	3	ha,hb	1	340	n	4	N	2	b?	n	17	80	f	75
62	47	s	50	15	3	ha,hb	1	340	n	3	W	2	b?	n	17	80	f	75

APPENDIX 9.Aa Field site data values

A	B	C	D	E	F	G	H	I	J
63	31a	Lewes, Cuilfail Tunnel Porta	TQ425100	Main	C	dq,rc	50	1	7
64	31b	Lewes, Cuilfail Tunnel Porta	TQ425100	S	C	dq,rc	18	3.5	7
65	32a	Middlebarrow Quarr	SD468768	Lower	L	dq	15	1	60
66	32b	Middlebarrow Quarr	SD468768	Middle	L	dq	60	0.5	60
67	32c	Middlebarrow Quarr	SD468768	Upper	L	dq	12	5	60
68	33a	Mossborough Park Way, Sheffiel	SK437846	S tabular	S	rc	25	0.3	70
69	33b	Mossborough Park Way, Sheffiel	SK437846	S blocky	S	rc	50	2	70
70	33c	Mossborough Park Way, Sheffiel	SK437846	S fissile	S	rc	10	1	10
71	33d	Mossborough Park Way, Sheffiel	SK437846	N	S	rc	60	1.5	70
72	34a	Nafferton Land Limes Quarr	TA047612	Wf	C	dq	6	2.5	47
73	34b	Nafferton Land Limes Quarr	TA047612	Nf	C	dq	6	2.5	47
74	34c	Nafferton Land Limes Quarr	TA047612	Ef	C	dq	6	2.5	47
75	35a	Orton, Appleby	NY597093	Upper	L	dq,rc	32	2.5	70
76	35b	Orton, Appleby	NY597093	Sf lower	L	dq,rc	80	10	70
77	35c	Orton, Appleby	NY597093	Ef lower	L	dq,rc	55	10	70
78	36a	Over Kellet Quarry, Carnfortl	SD516686	Upper	L	dq	15	2	70
79	36b	Over Kellet Quarry, Carnfortl	SD516686	Lower	L	dq	100	0.5	70
80	37	Pex Hill Quarry, Widnes	SJ505889		S	dq	60	0.2	45
81	38a	West Quantockshead, A3	ST113423	S 1	S	rc	40	0.5	40
82	38b	West Quantockshead, A3	ST113423	S 2	S	rc	8	0.5	40
83	38c	West Quantockshead, A3	ST113423	S 3	S	rc	25	0.5	40
84	38d	West Quantockshead, A3	ST113423	S 4	S	rc	50	0.2	4
85	38e	West Quantockshead, A3	ST114425	N	S	rc	25	1	80
86	39a	Redills Road Cut A66 (RIGS	NY504287	Upper	L	rc	25	3	80
87	39b	Redills Road Cut A66 (RIGS	NY504287	Lower	S	rc	50	1.5	50
88	40a	Runcorn Expressway, Merseysid	SJ506815	Main	S,M	rc	250	0.2	4
89	40b	Runcorn Expressway, Merseysid	SJ506815	Honeycomb	S,M	rc	250	0.2	15
90	41a	Runcorn Hill, Merseyside	SJ508817	Nf upper	S	dq	35	2	30
91	41b	Runcorn Hill, Merseyside	SJ508817	Nf lower	S	dq	150	1.5	30
92	41c	Runcorn Hill, Merseyside	SJ508817	Wf	S	dq	150	0.2	30
93	42	Sandiforth Farm OCCS	SD552014		Z	aq	15	2.5	40
94	43	Sedburgh Quarry	SD613988		S,Z,Sh	dq	25	4.5	50
95	44	Southerham Works Quarry, Lewes	TQ425095		C	rc,dq	10	2	20
96	45a	Stockhowall Quarr	NY067176	NWf	L	dq	40	0	100
97	45b	Stockhowall Quarr	NY067176	SEf	L	dq	20	2	100
98	46a	Stocksbridge Bypass	SK300989	E	S	rc	40	0.2	80
99	46b	Stocksbridge Bypass	SK300989	W	S	rc	80	0.5	80
100	47a	Sutton Bank, A70, N Yorkshire	SE515827	Upper	Sc	rc	30	0.5	32
101	47b	Sutton Bank, A70, N Yorkshire	SE515827	Massive	Sc	rc	50	2	32
102	47c	Sutton Bank, A70, N Yorkshire	SE515827	Bedded	Sc	rc	10	3	32
103	47d	Sutton Bank, A70, N Yorkshire	SE515827	Blocky	Sc	rc	25	3	32
104	48	Swinton, Malton, N Yorkshire	SE758730		Lo	dq	45	0.8	10
105	49a	Thurstaston Hill, Wirral	SJ243848	N massive	S	rc	125	0.5	20
106	49b	Thurstaston Hill, Wirral	SJ243848	N massive	S	rc	1	0.2	20
107	49c	Thurstaston Hill, Wirral	SJ243848	S massive	S	rc	125	0.5	20
108	49d	Thurstaston Hill, Wirral	SJ243848	S layerec	S	rc	1	0.2	20
109	50	Woodside Quarry	SE563383		G	saq	50	2	60
110	51	Yorkgate Quarry, East Chevi	SE198442		G	dq	120	3	60
111	52	Blair Atholl A, A9	NN777661	Sill	Gn	rc	7	2	180
112	53a	Bramcrag Quarry, Cumbria	NY320220	E 1	Gm	dq	70	0.5	200
113	53b	Bramcrag Quarry, Cumbria	NY320220	E 2	Gm	dq	15	5	200
114	53c	Bramcrag Quarry, Cumbria	NY320220	E 3	Gm	dq	30	2	200
115	54d	Bramcrag Quarry, Cumbria	NY320220	W	Gm	dq	30	2	200
116	55a	Cambusbarron Quarry, Stirling	NS770914	NWf	D,Bas	dq	35	10	120
117	55b	Cambusbarron Quarry, Stirling	NS770914	SWf	D,Bas	dq	120	20	120
118	56a	Caykavak, south central Turke	37.6°N34.5°E	1	Lp	rc	135	0.5	200
119	56b	Caykavak, south central Turke	37.6°N34.5°E	2	Lp	rc	50	0.2	150
120	56c	Caykavak, south central Turke	37.6°N34.5°E	3	Lp	rc	30	0.2	150
121	57	Doganli, south central Turkey	38.2°N34.9°E		P	n	250	2.5	10
122	58	Dunsinane Hill Quarr	NO208316		D	aq	40	0.2	150
123	59a	Eskdale Granite Quarr	NY164004	NEf	Gn	dq	250	1	250
124	59b	Eskdale Granite Quarr	NY164004	SEf	Gn	dq	90	1.5	250
125	60a	Glen Coe Milestone	NN186564	1	Bas	rc	200	0.4	220
126	60b	Glen Coe Milestone	NN186564	2	Bas,Peg	rc	5	1.5	220
127	60c	Glen Coe Milestone	NN186564	3	Peg	rc	25	1.5	220
128	60d	Glen Coe Milestone	NN186564	4	Bas	rc	10	0.8	220
129	60e	Glen Coe Milestone	NN186564	5	Peg	rc	40	1.5	220
130	60f	Glen Coe Milestone	NN186564	6	Bas,Peg	rc	40	0.5	220
131	60g	Glen Coe Milestone	NN186564	7	Bas	rc	15	0.2	220
132	61a	Glen Coe Study	NN183564	NW	Peg	rc	20	0.5	220
133	61b	Glen Coe Study	NN180564	Middle	Peg	rc	55	0.5	220
134	61c	Glen Coe Study	NN177564	SE	Peg	rc	65	0.5	220
135	62a	The Gobbins, Northern Irelanc	J469997	Upper	Bas	rc	80	0.2	180
136	62b	The Gobbins, Northern Irelanc	J469997	Lower	Bas	rc	130	0.2	180

APPENDIX 9.Aa Field site data values

A	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB
63	u	f+	20	60	5	c	2	40	n	2	W	1	m?	n	>50	88	f	>50?
64	u	f+	20	60	1	c	2	40	n	2	W	1	m?	n	17	85	f	<30?
65	u	f	20	15	2	c	1	50	y	1	N	e1	b	n	12	80	f	5
66	u	f	100	60	3	c	1	50	y	2	N	e1	b	n	12	80	f	5
67	u	f	20	12	2	c	1	50	y	2	N	e1	b	n	6	55	f	5
68	50-54	f+	100	25	5	sa	1	30	n	2	NW	e3	m,b	n	7	60-74	f	8
69	50-54	f+	100	50	5	c	1	30	n	2	NW	e3	m,b	n	7	60-74	f	8
70	u	h+	0	10	2	sa	1	30	n	2	NW	e3	m,b	n	7	60-74	f	8
71	50-54	f+	100	80	5	sa	1	30	n	2	SE	e2	m,b	n	6	45	f	8
72	44	f+	0	10	5	hb	1	80	n	1	W	2	m	n	5	85	f	13
73	44	f+	0	10	5	hb	1	80	n	1	N	2	m	n	5	85	f	13
74	44	f+	0	10	5	hb	1	80	n	1	E	2	m	n	6	85	f	>20?
75	55(L)	f	60	32	4	ha,sa	1	305	n	3	W	e2	b	n	6	85-90	f	>50?
76	55(L)	f	100	80	4	sa	1	305	n	2	S	e1	b	n	8	90	f	<10
77	55(L)	f	100	55	4	sa	1	305	n	2	E	e3	b	n	8	90	f	<10
78	u	f	10	15	3	sa	1	110	n	2	All	e3	b	n	15	90	f	?20
79	u	f	100	100	5	ha	1	110	n	2	All	e3	b	n	15	90	f	?20
80	u	s	100	60	5	c	1	50	n	1	All	3	h,b	n	7-25	85-88	f	>100
81	u	s	100	40	4	sa	1	130	n	2	SW	e1	h,b	n	8	68	f	20?
82	u	m+	0	25	4	sa	1	130	n	2	SW	e1	h,b	n	8	68	f	20?
83	u	s+	50	25	4	sa	1	130	n	2	SW	e1	h,b	n	8	68	f	20?
84	u	c	0	0	1	sb	1	130	n	1	SW	e1	h,b	n	8	52	f	20?
85	u	s	100	50	5	c	4	130	n	2	NW	1	b	n	7	68	u	20?
86	u	s	100	25	3	sa	1	150	n	2	SW	1	u	n	7	54	f	>20?
87	u	m	80	50	5	sa	1	150	n	2	SW	1	u	n	7	54	f	>20?
88	u	m	0	250	5	ha	1	70	y?	1	W	e2	m	n	8	72-82	f	<30?
89	u	s+	100	250	5	ha	1	70	y?	1	SE	e2	m	n	17	82	f	>50?
90	u	s+	50	35	5	sa	1	70	y?	1	N	e2	h	n	5	87	f	>100
91	u	s+	100	150	5	sa	1	70	y?	1	N	e2	h	n	15	87	f	>100
92	u	s	100	150	5	sa	1	70	y?	1	W	2	h	n	15	87	f	>100
93	39	f	0	40	2	ha	2	100	n	2	S	1	b	n	14	80	f	0
94	65	s	30	25	3	ha,hb	3	310	n	3	W	2	b	n	7	70	u	>30?
95	u	f+	0	15	4	sa	1	50	n	2	NW	3	m?	n	7	80	f	20
96	u	f+	100	40	2	c	1	190	n	1	NW	e3	b	n	14	75	f	>50?
97	u	f+	90	20	5	ha,sa	1	190	n	1	SE	e1	b	n	10	75	f	<30?
98	60	s	100	60	4	sa	1	200	n	2	S	e2	b,m	y	7	76	f	>30?
99	60	s	100	80	4	sa	1	200	n	2	S	e2	b,m	y	14	76	f	11
100	u	s+	90	30	5	sa	1	250	n	3	W	e1	m	n	10	55	f	9
101	u	s+	100	80	5	sa	2	250	n	3	W	e2	m	y	10	80	u	9
102	u	s+	20	30	2	sa,sb	1	250	n	3	W	e1	m	y	10	80	f	9
103	u	m	50	50	4	sa,sb	1	250	n	3	W	e2	m	y	10	80	f	9
104	30	s	0	45	5	sa	1	50	n	2	N	e2	u	n	9	84	f	>30?
105	u	s	100	125	5	sa	1	55	n	2	SW	1	h	n	6	87	f	>100?
106	u	s	100	125	5	sa	1	55	n	2	SW	1	h	n	6	87	f	>100?
107	u	s	100	125	5	sa	1	55	n	2	NE	e3	h	n	6	87	f	>100?
108	u	s	100	125	5	sa	1	55	n	2	NE	e3	h	n	6	87	f	>100?
109	u	m	80	50	5	sa,sb	1	110	n	2	All	e1	b,m	n	8	75	f	9
110	u	m	100	120	4	sa	1	270	n	3	S	e1	h?	n	7	90	f	>100
111	70	f	100	250	3	sa	1	250	n	3	W	e2	b(p)	n	8	88	f	u
112	70	f	100	70	4	c,sa	1	220	n	2	W	3	b	n	40	83	f	>30?
113	70	f	50	15	2	c,sa	1	220	n	2	W	3	b	n	40	83	f	>30?
114	70	f	100	30	3	c,sa	1	220	n	2	W	3	b	n	20	83	f	>30?
115	70	f	100	30	3	c,sa	1	220	n	1	E	3	b	n	7	83	f	>30?
116	62	f+	100	35	2	sb	1	120	n	2	NW	3	b	n	18	85	f	?>50
117	62	f+	100	120	5	hb	1	120	n	2	SW	3	b	n	18	85	f	?>50
118	u	f+	100	135	2	sa	2	1600	n	3	W	1	b	n	15	80	f	15
119	u	s	100	50	2	sa	2	1600	n	3	E	1	b	n	10	70	f	15
120	u	s	100	30	2	sa	2	1600	n	3	W	1	b	n	15	80	f	15
121	u	m	0	250	3	ha	1	1540	n	2	S	e1	n	n	10	80	f	n
122	u	f	100	40	4	ha	1	220	n	3	SW	1	b	n	13	75	f	0
123	65-70	f	100	250	5	ha	4	60	n	1	NE	3	b	n	15	80-85	f	>50?
124	65-70	f	100	90	5	ha	1	60	n	1	SW	e1	b	n	35	80-85	u	>50?
125	72	s	100	200	5	c	1	260	n	3	NE	e2	b	n	9	82	f	>50?
126	70-72	f	0	5	4	sa	1	260	n	3	NE	4	b	n	9	82	f	>50?
127	70	f	20	90	3	sa	1	260	n	3	NE	3	b	n	9	82	f	>50?
128	72	f	0	10	5	sa	1	260	n	3	NE	1	b	n	9	82	f	>50?
129	70	f	80	60	5	c,ha	1	260	n	3	NE	1	b	n	9	82	f	>50?
130	70-72	f	100	40	4	sa	2	260	n	3	NE	2	b	n	9	82	u	>50?
131	72	f	40	50	3	sa	2	260	n	3	NE	1	b	n	9	82	u	>50?
132	70	f	90	20	2	sa	1	260	n	3	SW	1	b	n	10	78	u	>100?
133	70	f	100	55	4	sa	1	260	n	3	SW	1	b	n	10	78	u	>100?
134	70	f	100	55	4	sa	1	260	n	1	S	1	b	n	16	85	u	>100?
135	u	s+	100	80	4	c	2	65	y	4	E	1	u	n	8	82-90	f	40?
136	u	s+	100	130	4	c	2	65	y	4	E	1	u	n	8	82-90	f	40?

APPENDIX 9.Aa Field site data values

A	B	C	D	E	F	G	H	I	J
137	63a	Hilltop Quarry, Cumbria	NY320230	1	Gm	dq	40	5	200
138	63b	Hilltop Quarry, Cumbria	NY320230	2 planar a	Gm	dq	20	6	200
139	63c	Hilltop Quarry, Cumbria	NY320230	2 planar b	Gm	dq	45	3	200
140	63d	Hilltop Quarry, Cumbria	NY320230	2 c	Gm	dq	45	10	200
141	63e	Hilltop Quarry, Cumbria	NY320230	3	Gm	dq	10	5	200
142	64a	Knock Pike Quarry, Appleby	NY687286	1	I	dq	15	7	150
143	64b	Knock Pike Quarry, Appleby	NY687286	2	I	dq	15	7	150
144	64c	Knock Pike Quarry, Appleby	NY687286	3	I	dq	15	7	150
145	65a	Knotts Quarry, Cumbria	NY432217	1	V	dq	150	0.8	220
146	65b	Knotts Quarry, Cumbria	NY432217	2	V	dq	30	1	220
147	66a	Near White Moss Quarry, Cumbria	NY345064	1	Tf	dq	80	0.5	70
148	66b	Near White Moss Quarry, Cumbria	NY345064	2	Tf	dq	30	1.5	70
149	67	Quayfoot Quarry, Cumbria	NY252167		Tf	dq	45	1	200
150	68	Thrang Crag Quarry, Cumbria	NY316056		Tf	dq	60	1	70
151	69a	Threlkeld Quarry, Keswick	NY328242	1	Gm	dq	30	3.5	60
152	69b	Threlkeld Quarry, Keswick	NY328242	2	Gm	dq	150	0.7	60
153	70	Torver Quarry	SD289909		Gm	dq	30	0.1	220
154	71a	White Moss Quarry	NY348065	Upper	Tf	dq	35	0.2	100
155	71b	White Moss Quarry	NY348065	Lower	Tf	dq	100	0.2	100
156	71c	White Moss Quarry	NY348065	NE 1	Tf	dq	150	0.2	100
157	71d	White Moss Quarry	NY348065	NE 2	Tf	dq	35	0.2	100
158	72a	Banavie Quarry, Corpach	NN113775	Dyke	D	aq	2	0.1	25
159	72b	Banavie Quarry, Corpach	NN113775	Felsite	Gf	aq	15	2.5	130
160	72c	Banavie Quarry, Corpach	NN113775	Gneiss	X,Sch	aq	25	1	130
161	72d	Banavie Quarry, Corpach	NN113775	Old	Gf,X	dq	20	1.5	160
162	73a	Beck Wythop, Cumbria	NY216281	N planar	T	rc	50	0.2	140
163	73b	Beck Wythop, Cumbria	NY216281	N irreg.	T	rc	50	0.2	140
164	73c	Beck Wythop, Cumbria	NY216281	S	T	rc	50	0.2	140
165	74a	Blair Atholl B, A9	NN777661	1	X	rc	8	0.5	120
166	74b	Blair Atholl B, A9	NN777661	2	X	rc	80	1	120
167	74c	Blair Atholl B, A9	NN777661	3	X	rc	20	2	120
168	74d	Blair Atholl B, A9	NN777661	4	X	rc	10	4	120
169	75a	Corpach A830, north side	NN078772	Main	Sch	rc	15	1.5	100
170	75b	Corpach A830, north side	NN078772	Dyke	Peg	rc	4	1.5	60
171	76	Crianlarich A82(T)	3825		Sch	rc	150	0.2	220
172	77	Daviot A9	NH737344		X	rc	22	2	60
173	78	Doolough Pass Road, Ireland	L844645		S,Z	rc	6	1.5	30
174	79	Haverthwaite A590(T	SD344843		T	rc	15	0.1	100
175	80	Hodge Close	NY318018		Sl	dq	250	0	150
176	81a	Hodgson How Quarry, Keswick	NY244237	1	Sl	dq	5	0	180
177	81b	Hodgson How Quarry, Keswick	NY244237	2	Sl	dq	5	0	180
178	81c	Hodgson How Quarry, Keswick	NY244237	3	Sl	dq	5	0	180
179	82a	Killiecrankie Pass A9	NN915613	1	X,Sch	rc	250	0.1	180
180	82b	Killiecrankie Pass A9	NN915613	2	X,Sch	rc	90	1.5	180
181	82c	Killiecrankie Pass A9	NN915613	3	X,Sch	rc	15	2.5	50
182	82d	Killiecrankie Pass A9	NN915613	4	X,Sch	rc	30	2	110
183	83a	Knotts Quarry, Cumbria	NY432217	1	T	dq	30	3.5	160
184	83b	Knotts Quarry, Cumbria	NY432217	2	T	dq	1	0	160
185	83c	Knotts Quarry, Cumbria	NY432217	3	T	dq	250	0	160
186	84a	Lindale A590	SD418807	1	T	rc	15	0.5	70
187	84b	Lindale A590	SD418807	2	T	rc	45	2	100
188	84c	Lindale A590	SD418807	3	T	rc	8	1.5	90
189	85a	Loch Lomond	NS355220	1	X	rc	100	0.2	200
190	85b	Loch Lomond	NS355220	2	X	rc	15	2.5	200
191	86a	Loch Lomond I	NS357903	1	X,Phyl	rc	20	0.1	150
192	86b	Loch Lomond I	NS357908	2	X,Phyl	rc	2	0.5	25
193	87a	Lune Gorge A685	NY607010	1	T	rc	30	5	50
194	87b	Lune Gorge A685	NY607010	2	T	rc	12	0.1	80
195	88a	Lune Gorge M6, Dillicar	NY610002	Layered	T	rc	35	0.4	140
196	88b	Lune Gorge M6, Dillicar	NY610002	Fissile	T	rc	10	1.5	50
197	89	Lune Gorge M6, Borrowdale	NY607010		T	rc	20	2.5	140
198	90	Lune Gorge M6, Jeffrey's Moun	NY609024		T	rc	70	0.5	140
199	90a	Scawgill Bridge Quarry	NY177258	Ef	T	dq	40	0	120
200	90b	Scawgill Bridge Quarry	NY177258	Sf	T	dq	40	0	120
201	91	School House Quarry	NY364306		Sl	dq	20	1	200
202	92	Skiddaw Slate Quarry	NY229202		Sl	dq	10	0	180
203	93	Slochd Summit A9	NH837255		X,F	rc	12	3	70
204	94	Stybarrow Crag	NY386179		Sl	rc	100	0.1	150
205	95a	Temple Pier	NH544308	Main	X	rc	30	0.1	160
206	95b	Temple Pier	NH544308	Sill	D	rc	12	0.1	150
207	95c	Temple Pier	NH544308	Lower	X	rc	120	0.1	180
208	96	Torver Quarry I	SD289909		Sl	dq	8	0.2	50
209	97a	Wasdale Beck A6	NY555077	Fissile	T	rc	10	0.1	120
210	97b	Wasdale Beck A6	NY555077	Massive	T	rc	80	0.1	120

APPENDIX 9.Aa Field site data values

A	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB
137	70	f+	100	40	3	sa	2	250	n	2	S	1	b	n	12	66	u	?30
138	70	f+	80	20	3	sa	1	250	n	2	SW	1	b	n	12	81	f	?30
139	70	f+	100	45	3	sa	1	250	n	2	SW	1	b	n	12	81	f	?30
140	70	f+	100	45	5	sa	1	250	n	2	SW	1	b	n	12	81	f	?30
141	70	f+	60	15	4	sa	1	250	n	2	W	1	b	n	15	50-65	f	?30
142	66	s	100	30	2	c	1	335	n	2	SE	1	b	n	30	54	f	?20
143	66	s	100	30	2	c	1	335	n	2	NE	1	b	n	30	54	f	?20
144	66	s	100	30	2	c	1	335	n	2	NW	1	b	n	30	54	f	?20
145	60	f+	100	150	5	c	1	250	n	1	SE	e2	b	n	18	82	f	?>50
146	60	f+	90	30	1	c	1	250	n	1	SE	e2	b	n	18	82	f	?>50
147	67	f+	100	80	5	c	3	100	n	2	E	e2	u	n	9	95	f	?>50
148	67	f+	90	30	5	c	1	100	n	1	N	e3	u	n	3	68	f	>80?
149	50	f	100	45	5	ha,sa	4	110	n	1	NW	2	u	n	20	115	u	>100
150	62	f	50	60	4	c	3	140	n	2	SE	3	u	n	?13	90-110	u	?20
151	55-60	s+	100	30	3	ha,hb,sa	1	240	n	1	N-NW	1	b	n	15	50	f	18
152	55-60	s+	100	150	5	ha,hb	1	240	n	1	N-NW	1	b	n	15	50	f	18
153	68-72	f+	100	30	4	c,sa	1	65	n	1	NE	4	b	n	6	90	f	?>50
154	64	f+	90	35	5	c,sa	3	60	n	1	SE	4	b?m	n	14	95	f	>50?
155	64	f+	100	100	5	c,sa	3	60	n	1	SE	4	b?m	n	14	95	f	>50?
156	64	f+	100	150	5	c,sa	1	60	n	1	SE	3	b?m	n	14	95	f	>50?
157	64	f+	90	35	5	c,sa	2	60	n	1	SW	4	b?m	n	14	85	f	>50?
158	u	m	0	250	1	c	1	80	n	3	S	1	b	n	15-2C	70	f	0
159	60	f	70	15	3	c	1	80	n	3	S	1	b	n	15-2C	70	f	0
160	70	f	90	25	3	c	1	80	n	3	S	1	b	n	15-2C	70	f	0
161	65	s	80	20	3	c	1	80	n	2	N	e2	b	n	16-2C	84	f	20?
162	54-70	s	100	50	5	c,ha,sa	1	100	n	2	NE	2	u	n	16	80	f	>50?
163	54-70	s	100	50	5	c,ha,sa	1	100	n	2	NE	2	u	n	16	80	f	>50?
164	54-72	s	100	50	5	c,ha,sa	1	100	n	2	NE	2	u	n	16	80	f	>50?
165	66-68	f	0	60	2	ha,sa	1	250	n	3	W	1	pb	n	14	65-82	f	?20
166	66-68	f	100	80	5	ha,sa	1	250	n	3	W	1	pb	n	20	65-82	f	?20
167	66-68	f	90	20	5	ha,sa	1	250	n	3	W	1	pb	n	14	65-82	f	?20
168	66-68	f	90	10	5	ha,sa	1	250	n	3	W	1	pb	n	14	65-82	f	?20
169	70?	s	50	20	5	c	1	15	n	2	SW	1	pb?	n	8	50	f	?30
170	u	s	0	0	1	ha	1	15	n	2	SW	1	pb?	n	8	50	f	?30
171	54-66	f	100	150	2	sa	1	220	n	3	SE	3	pb?	y	7	75	f	30-50?
172	68	f	100	22	5	sa	1	340	n	2	S	4	b	y	15	72	f	20?
173	u	s+	0	6	2	sa	3	40	n	3	S	e1	b	n	7	90-110	u	>30?
174	53-63	s+	10	15	3	sa	1	20m	n	2	S	e2	b?	n	11	80	f	>50?
175	62	s	100	250	5	sa	3	150	n	1	All	4	b?	n	30	90	f	>100
176	61	s	100	50	5	c	3	85	n	1	S	1	b?	n	14	85-90	f	>100
177	61	s	100	50	5	c	1	85	n	1	SW	4	b?	n	14	85-90	f	>100
178	61	s	100	50	5	c	1	85	n	1	NW	3	b?	n	14	85-90	f	>100
179	u	f+	100	250	5	ha	1	170	n	4	W-NW	2	pb	n	18	80	f	30-50?
180	u	f+	100	90	4	ha	1	170	n	4	W-NW	2	pb	y	18	80	f	30-50?
181	u	f+	30	30	3	ha	1	170	n	4	W-NW	2	pb	y	18	80	f	30-50?
182	u	f+	70	30	3	ha	1	170	n	4	W-NW	2	pb	n	18	80	u	30-50?
183	54	f+	100	30	2	c	1	250	n	1	SE	e2	b	n	18	82	f	?>50
184	54	f+	0	250	3	c	1	250	n	1	SE	e2	b	n	18	82	f	?>50
185	54	f+	100	250	4	c	3	250	n	1	SE	e2	b	n	18	82	u	?>50
186	u	s+	30	15	3	c	1	10	n	2	SE	e1	u	n	10	75	f	30-50?
187	u	f+	100	45	4	sa	2	10	n	2	SE	e1	u	n	20	50	f	30-50?
188	u	s	0	8	1	ha	1	10	n	2	SE	e2	u	n	20	50	f	30-50?
189	66	f+	100	100	5	c	1	30	n	2	E	e3	b	n	12	36	f	30-50?
190	66	f+	20	100	2	c	1	30	n	2	E	3	b	n	12	36	f	30-50?
191	70	f	80	80	5	c,ha	1	30	n	2	E	1	pb?	n	14	55	f	30-50?
192	45?	s	0	0	1	c,ha	1	30	n	2	E	1	pb?	n	14	55	f	30-50?
193	u	f+	30	30	4	sa	1	250	n	4	E	1	pb	y	14	78	f	29
194	u	f+	0	12	2	sa	1	250	n	4	E	2	pb	y	13	78	f	29
195	u	f+	80	35	4	sa	1	200	n	4	NE	3	pb	y	11	64	f	29
196	u	f+	0	10	3	sa	1	200	n	4	NE	3	pb	y	11	64	f	29
197	u	f+	30	20	4	sa	2	210	n	4	E	1	pb	y	8	64	f	29
198	u	f+	70	70	4	sa	3	240	n	4	SE	2	pb	y	14	75	u	29
199	68	f	100	40	5	c	1	200	n	3	E	1	b	n	20	90	f	>40
200	68	f	100	40	5	c	1	200	n	3	S	3	b	n	20	90	f	>40
201	62	f	100	40	5	ha,sa	1	230	n	2	S	1	u	n	10	76	f	>100?
202	61	s	100	50	5	ha	1	180	n	1	SW	4	b?	n	20	85-90	f	>100
203	58-64	f+	20	30	3	ha,hb	2	400	n	3	SW	3	b?	n	12	54	f	<20?
204	u	f+	100	100	5	c	3	160	n	4	E	1	u	y	20?	90	f	>80
205	68	f	100	30	5	sa	1	52	n	3	SE	2	b?	y	16	75	f	<20?
206	u	f	30	200	3	c	1	52	n	3	SE	2	b?	n	13	75	f	<20?
207	u	f	100	120	5	c	1	52	n	3	SE	1	b?	y	6	75	f	<20?
208	39-42	s	10	8	2	c,ha	1	65	n	1	SE	2	b	n	9	90	f	?>50
209	52	f+	0	10	3	sa	1	380	n	4	NW	e1	b	n	6	85	f	>100
210	52	f+	100	80	4	sa	1	380	n	4	NW	e2	b	n	6	85	f	>100

APPENDIX 9.Aa Field site data values

Site number	Fracture spacing	Fracture aperture	Rock strength	Weathering grade	RMR rock strength	RMR RQD	RMR fracture spacing	RMR fracture aperture	RMR wall weathering	RMR persistence	RMR fracture infilling	RMR total fracture condition	RMR groundwater seepage	RMR favourability	Adjustment A1.a	Adjustment A1.b	Adjustment A1.c	Adjustment A1.d	Adjustment A2.a	Adjustment A2.b	Adjustment B	Adjustment C	Adjustment D2	Adjustment E1.a	Adjustment E1.b	Adjustment G (engineering)	Adjustment H1.a	Adjustment H1.b
	RDA data				RMR data										Environment								Stress					
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC
1	32	2	30	10	2	3	5	4	3	2	2	13.75	15	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0
2	15	8	12	5	6	20	12	1	5	0	2	10	10	0	0	0	2	0	0	0	2	1	0	0	0	0	0	0
3	5	6	12	5	6	20	18	1	5	0	2	10	10	0	0	0	2	0	0	0	2	1	0	0	0	0	0	0
4	10	2	15	5	5	20	14	4	5	0	2	13.75	8	0	0	2	0	0	0	0	1	2	0	0	0	0	0	0
5	12	7	15	5	5	20	14	1	5	1	2	11.25	8	0	0	2	0	0	0	0	1	2	0	0	0	0	0	0
6	15	2	15	5	5	18	14	4	5	0	2	13.75	8	0	0	2	0	0	0	0	1	2	0	0	0	0	0	0
7	12	7	15	5	5	20	14	1	5	0	2	10	12	0	0	2	0	0	0	2	0	1	0	0	0	0	0	0
8	10	2	15	5	5	20	14	4	5	0	2	13.75	8	0	0	2	0	0	0	0	1	2	0	0	0	0	0	0
9	15	2	15	5	5	20	14	4	5	0	2	13.75	8	0	0	2	0	0	0	1	0	2	0	0	0	0	0	0
10	15	2	15	5	5	20	14	4	5	0	2	13.75	8	0	0	2	0	0	0	0	1	2	0	0	0	0	0	0
11	21	6	8	3	8	10	7	1	6	0	0	8.75	7	0	0	6	0	0	0	0	1	2	0	0	2	0	0	0
12	18	2	8	2	8	20	10	4	6	0	3	16.25	7	0	0	6	0	0	0	0	1	2	0	0	2	0	0	0
13	25	6	8	0	8	10	12	1	6	0	3	12.5	7	0	0	6	0	0	0	0	1	2	0	0	2	0	0	0
14	22	11	13	12	6	16	11	0	3	0	5	10	10	0	0	0	0	0	0	0	0	1	1	0	3	3	0	0
15	8	9	13	11	6	20	18	1	3	0	5	11.25	10	0	0	0	0	0	0	0	0	1	1	0	3	3	0	0
16	0	1	33	10	1	0	20	4	3	0	6	16.25	7	0	0	0	0	0	1	0	0	2	1	0	0	0	0	0
17	5	2	22	10	3	20	18	4	3	1	2	12.5	5	0	0	0	0	0	1	0	0	3	1	0	0	0	0	0
18	23	2	15	12	5	10	8	4	3	2	2	13.75	5	0	0	0	0	0	1	0	0	3	1	0	0	0	0	0
19	5	2	22	10	3	20	18	4	3	1	2	12.5	5	0	0	0	0	0	0	0	0	3	2	0	0	0	0	0
20	23	2	15	12	5	10	8	4	3	2	2	13.75	5	0	0	0	0	0	0	0	0	3	2	0	0	0	0	0
21	34	2	22	10	3	3	5	4	2	4	2	15	5	0	0	0	0	0	0	0	0	3	1	0	0	0	0	0
22	3	2	14	5	5	20	20	4	5	0	2	13.75	13	0	0	2	0	0	0	0	0	1	0	0	0	0	0	0
23	28	7	7	2	10	5	6	1	6	1	1	11.25	14	0	0	0	0	3	0	0	0	1	0	0	0	0	0	0
24	12	0	6	0	11	20	13	6	6	2	4	22.5	13	0	0	0	0	3	0	0	1	3	0	0	0	0	0	0
25	10	4	24	12	2	20	14	1	2	1	3	8.75	6	0	0	2	0	0	0	0	3	2	0	0	2	0	0	0
26	0	6	32	3	1	0	20	1	5	0	2	10	10	0	0	0	0	0	1	0	0	2	1	0	0	0	0	0
27	1	13	8	1	8	20	20	0	6	0	5	13.75	13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
28	10	9	8	1	8	20	14	1	6	0	5	15	10	0	0	0	0	0	0	0	2	2	0	0	0	0	0	0
29	16	6	8	1	8	20	11	1	6	0	5	15	13	-50	0	0	0	0	0	1	0	1	0	0	0	0	0	0
30	16	6	8	1	8	20	11	1	6	0	5	15	10	0	0	0	0	0	0	0	3	2	0	0	0	0	0	0
31	16	6	8	1	8	20	11	1	6	0	5	15	10	0	0	0	0	0	0	0	1	2	0	0	0	0	0	0
32	2	10	22	6	3	20	20	1	4	1	2	10	8	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0
33	2	10	22	6	3	20	20	1	4	1	2	10	8	0	0	0	0	0	0	0	0	3	2	0	0	0	0	0
34	26	6	7	3	10	10	9	1	6	2	2	13.75	4	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0
35	23	5	7	3	10	20	14	1	6	1	2	12.5	4	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0
36	5	1	15	5	4	20	18	4	5	0	2	13.75	10	0	0	0	3	0	0	0	2	1	0	0	0	0	0	0
37	10	10	15	6	4	16	15	1	6	0	6	16.25	12	0	0	0	3	0	0	0	1	1	0	0	0	0	0	0
38	35	4	32	14	1	0	5	1	1	6	4	15	9	0	0	0	1	0	0	0	1	3	2	0	0	0	-2	0
39	13	5	14	3	5	16	13	1	6	2	5	17.5	15	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
40	19	1	8	3	8	18	10	4	6	0	5	18.75	12	0	0	5	0	0	0	0	1	1	0	0	0	0	0	0
41	13	1	8	2	8	20	13	4	6	0	6	20	7	0	0	0	0	2	0	0	1	3	0	0	0	0	0	0
42	20	8	8	2	8	18	10	1	6	1	4	15	10	0	0	0	0	2	0	0	1	1	0	0	0	0	0	0
43	28	10	8	5	8	5	5	1	5	6	3	18.75	10	0	0	0	0	2	0	0	1	2	0	0	0	0	0	0
44	29	1	30	3	2	0	9	4	6	2	2	17.5	13	0	0	4	0	0	0	0	2	0	0	0	0	0	0	0
45	31	1	30	2	2	0	11	4	6	2	2	17.5	13	-25	0	4	0	0	0	0	2	0	0	0	0	0	0	0
46	35	2	32	14	1	0	5	4	1	6	5	20	7	0	0	0	0	0	0	0	0	4	0	0	1	0	-2	0
47	28	6	13	7	6	0	7	1	5	1	4	13.75	12	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
48	10	4	13	5	6	20	13	1	5	0	4	12.5	12	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
49	23	6	13	5	6	16	13	1	5	0	4	12.5	12	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
50	22	6	12	1	6	20	9	1	6	2	4	16.25	15	0	0	0	0	0	0	0	2	0	0	0	1	0	0	0
51	29	4	18	7	4	7	7	4	5	1	2	15	10	0	0	0	0	0	0	0	1	1	2	0	0	0	0	0
52	10	2	9	5	8	20	14	4	5	1	2	15	10	-25	0	0	0	0	0	0	1	1	2	0	0	0	0	0
53	24	8	13	3	6	7	8	1	6	2	3	15	10	-50	0	3	0	0	0	0	2	2	0	0	0	0	0	0
54	24	4	12	1	6	16	12	4	6	1	4	18.75	8	0	0	0	0	0	0	0	1	2	0	0	0	0	0	0
55	18	2	9	3	8	20	12	4	6	0	2	15	8	0	0	0	0	0	0	0	1	2	0	0	0	0	0	0
56	0	14	6	1	11	20	20	0	6	0	5	13.75	10	0	0	0	0	0	0	0	1	2	0	0	0	0	0	0
57	28	9	10	5	7	0	7	1	6	2	1	12.5	10	0	0	0	0	0	0	0	3	2	0	0	0	0	0	0
58	17	11	10	10	7	16	13	0	3	2	1	7.5	10	0	0	0	0	0	0	0	3	2	0	0	0	0	0	0
59	23	8	10	7	7	10	8	1	5	1	1	10	10	0	0	0	0	0	0	0	3	2	0	0	0	0	0	0
60	10	6	12	11	6	20	14	1	3	0	2	7.5	12	0	0	0	2	0	0	0	0	1	0	0	0	0	0	0
61	27	9	13	5	6	10	6	1	5	2	3	13.75	13	0	0	7	0	0	0	0	3	2	0	0	0	0	0	0
62	27	9	13	5	6	10	6	1	5	2	3	13.75	13	0	0	4	0	0	0	0	0	2	0	0	0	0	0	0

APPENDIX 9.Ab RDA and RMR Ratings for field parameters

Site number	Adjustment H1.c	Adjustment H.2	Adjustment H3.a	Adjustment H3.b	Adjustment H3.c	Adjustment J1.a and J1.c	Adjustment J1.b	Adjustment J2.a	Adjustment J2.b	Adjustment K2.a	Adjustment K2.b	Adjustment K3	Adjustment K4	Adjustment L	Adjustment M	RDA total mass rating	RDA total material rating	Unadjusted RDA Rating	Total adjustment excluding K1	Adjustment for K1	Total adjustment	Adjusted RDA Rating	Rock Mass Rating (RMR)	RMR adjusted for slopes
	Excavated slope													Other		RDA Totals							RMR	
A	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA
1	0	0	0	0	0	0	0	1	0	-3	0	0	0	-2	0	34	40	74	-2	0	-2	72	39	39
2	0	0	0	0	0	0	0	0	0	-2	0	0	0	-10	1	23	17	40	-6	0	-6	34	58	58
3	0	0	0	0	0	0	0	0	-3	0	0	-4	0	-10	0	11	17	28	-12	0	-12	16	64	64
4	0	0	1	0	0	0	0	0	-2	-6	0	0	0	-3	0	12	20	32	-5	0	-5	27	61	61
5	0	0	1	0	0	0	0	0	0	-1	0	0	0	-3	0	19	20	39	2	0	2	41	58	58
6	0	0	2	0	0	0	0	0	-1	-4	0	0	0	-3	0	17	20	37	-1	0	-1	36	59	59
7	0	0	0	5	0	2	0	0	0	-1	0	0	0	-3	0	19	20	39	8	0	8	47	61	61
8	0	0	2	0	0	2	0	0	0	-6	0	0	0	-3	0	12	20	32	0	0	0	32	61	61
9	0	0	0	0	0	0	0	0	-3	-6	0	0	0	-3	0	17	20	37	-7	0	-7	30	61	61
10	0	0	0	0	0	0	0	0	-3	-6	0	0	0	-3	0	17	20	37	-7	0	-7	30	61	61
11	0	0	2	0	0	-2	0	0	0	-6	0	0	0	-6	0	27	11	38	-1	0	-1	37	41	41
12	0	0	0	0	0	-2	0	0	0	-5	0	0	0	-6	0	20	10	30	-2	0	-2	28	61	61
13	0	0	0	0	0	0	-5	0	-1	0	0	0	0	0	0	31	8	39	5	0	5	44	50	50
14	0	0	0	0	3	0	0	0	0	0	0	0	0	-2	0	33	25	58	9	0	9	67	53	53
15	0	0	0	0	3	0	0	0	0	0	0	0	0	-2	0	17	24	41	9	0	9	50	65	65
16	0	0	0	0	3	0	0	0	0	0	0	3	0	0	0	1	43	44	10	13	23	67	44	44
17	0	0	x1	x3	4	0	0	0	0	0	0	2	0	-8	0	7	32	39	3	0	3	42	59	59
18	0	0	x1	x3	4	0	0	0	0	0	0	2	0	-8	0	25	27	52	3	0	3	55	42	42
19	0	0	x1	5	x2	0	0	0	0	0	0	2	0	-8	0	7	32	39	4	0	4	43	59	59
20	0	0	x1	5	x2	0	0	0	0	0	0	2	0	-8	0	25	27	52	4	0	4	56	42	42
21	0	0	x1	x3	4	0	0	0	0	0	0	2	0	-8	0	36	32	68	2	0	2	70	31	31
22	0	0	1	x1	0	4	0	0	0	-5	0	0	0	-3	0	5	19	24	0	0	0	24	72	72
23	0	0	x1	0	4	0	0	0	0	0	0	0	0	-1	2	35	9	44	9	0	9	53	46	46
24	0	0	1	0	0	-3	0	0	0	-3	0	0	0	-10	2	12	6	18	-6	0	-6	12	80	80
25	0	0	0	0	0	0	0	0	0	-2	0	1	1	0	2	14	36	50	11	0	11	61	51	51
26	0	0	0	0	2	0	0	0	-2	-2	0	0	0	-9	0	6	35	41	-7	0	-7	34	41	41
27	0	0	1	0	0	0	0	0	-2	0	0	0	0	0	0	14	9	23	-1	0	-1	22	75	75
28	0	0	x2	3	0	0	0	0	0	0	0	6	0	0	0	19	9	28	13	0	13	41	67	67
29	0	0	x2	3	0	0	0	0	0	0	0	0	2	0	0	22	9	31	7	0	7	38	67	17
30	0	0	x2	3	0	0	0	0	-3	0	0	0	0	0	0	22	9	31	5	0	5	36	64	64
31	0	0	x2	3	0	0	0	0	-3	0	0	0	0	0	0	22	9	31	3	0	3	34	64	64
32	0	0	0	0	x1	-2	0	0	0	-4	0	0	0	-10	0	12	28	40	-14	0	-14	26	61	61
33	0	0	0	0	3	-2	0	0	0	-3	0	0	0	-10	0	12	28	40	-7	0	-7	33	61	61
34	0	0	x1	3	x2	0	0	0	0	0	0	7	0	0	0	32	10	42	14	0	14	56	47	47
35	0	0	x1	3	x2	0	0	0	0	0	0	5	0	0	0	28	10	38	12	0	12	50	61	61
36	0	0	1	0	0	1	0	0	-3	-3	0	0	0	-10	0	6	20	26	-8	0	-8	18	66	66
37	0	0	1	0	0	2	0	0	0	0	0	0	0	0	4	20	21	41	12	0	12	53	63	63
38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	39	46	85	5	0	5	90	30	30
39	0	0	x2	3	x2	2	-2	0	0	0	0	3	0	0	0	18	17	35	7	0	7	42	67	67
40	0	0	0	0	0	0	0	0	0	-2	0	0	0	-3	1	20	11	31	3	0	3	34	67	67
41	0	0	0	0	0	0	-5	0	0	-7	0	0	0	-5	3	14	10	24	-8	0	-8	16	68	68
42	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	28	10	38	6	0	6	44	61	61
43	0	0	0	5	0	0	0	0	0	0	0	0	0	0	0	38	13	51	10	0	10	61	47	47
44	0	0	0	0	0	2	0	0	0	-4	0	0	0	0	0	30	33	63	4	0	4	67	42	42
45	0	0	x3	4	0	2	0	0	0	-4	0	0	3	-3	0	32	32	64	8	0	8	72	44	19
46	0	0	0	0	0	0	0	0	0	0	0	-5	0	-8	5	37	46	83	-5	0	-5	78	33	33
47	0	0	1	0	2	0	0	0	0	0	0	0	0	-1	0	34	20	54	3	0	3	57	39	39
48	0	0	1	0	2	0	0	0	0	0	0	0	0	-1	0	14	18	32	3	0	3	35	64	64
49	0	0	1	0	2	0	0	0	0	0	0	-3	0	-1	0	29	18	47	0	0	0	47	60	60
50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	28	13	41	4	0	4	45	66	66
51	0	0	0	0	0	0	0	0	0	-4	0	0	0	-5	0	33	25	58	-5	0	-5	53	43	43
52	0	0	0	0	0	0	0	0	0	0	0	0	2	-5	0	12	14	26	1	0	1	27	67	42
53	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	32	16	48	10	0	10	58	46	-4
54	0	0	x1	x2	2	0	0	0	0	-2	0	0	0	0	-2	28	13	41	1	0	1	42	61	61
55	0	0	x1	x2	4	0	0	0	0	0	0	7	0	0	-2	20	12	32	12	0	12	44	63	63
56	0	0	0	0	2	0	0	0	-3	-5	0	0	0	-10	3	14	7	21	-10	0	-10	11	75	75
57	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	37	15	52	7	0	7	59	37	37
58	0	0	3	x2	0	0	0	0	0	0	0	4	0	0	0	28	20	48	12	0	12	60	54	54
59	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	31	17	48	7	0	7	55	45	45
60	0	0	0	1	0	0	0	0	0	0	0	0	0	-10	2	16	23	39	-4	0	-4	35	60	60
61	0	0	1	0	0	2	0	0	0	0	0	2	0	-5	0	36	18	54	12	0	12	66	49	49
62	0	0	1	0	0	2	0	0	-2	0	0	2	0	-5	0	36	18	54	4	0	4	58	49	49

APPENDIX 9.Ab RDA and RMR Ratings for field parameters

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC
63	17	4	29	3	1	5	11	1	6	0	6	16.25	15	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
64	23	8	29	3	1	5	12	1	6	6	6	23.75	15	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
65	26	4	12	0	6	5	8	4	6	4	6	25	13	0	0	0	0	0	0	0	2	0	1	0	0	0	0	0
66	15	2	12	0	6	20	13	4	6	2	6	22.5	13	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
67	27	10	12	0	6	5	7	1	6	4	6	21.25	13	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
68	23	1	10	2	7	20	8	4	6	0	2	15	7	0	0	0	0	0	0	0	2	2	1	0	2	0	0	0
69	17	6	10	2	7	20	11	1	6	0	6	16.25	7	0	0	0	0	0	0	0	2	2	1	0	2	0	0	0
70	28	4	28	15	2	0	7	1	1	4	2	10	7	0	0	0	0	0	0	0	2	3	1	0	2	0	0	0
71	15	5	10	2	7	20	13	1	6	0	2	11.25	10	0	0	0	0	0	0	2	0	1	0	0	2	0	0	0
72	31	7	14	2	6	0	6	1	6	0	4	13.75	10	0	0	0	0	0	0	3	0	2	0	0	0	0	0	0
73	31	7	14	2	6	0	6	1	6	0	4	13.75	10	0	0	0	0	0	0	0	0	2	3	0	0	0	0	0
74	31	7	14	2	6	0	6	1	6	0	4	13.75	10	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0
75	20	7	10	0	7	12	9	1	6	1	3	13.75	10	0	2	0	0	0	0	0	0	2	0	0	0	0	0	0
76	12	13	10	0	7	20	13	0	6	1	2	11.25	15	1	1	0	0	0	0	0	0	2	0	0	0	0	0	0
77	16	13	10	0	7	20	11	0	6	1	2	11.25	9	1	1	0	0	0	0	0	0	4	0	0	0	0	0	0
78	26	6	10	1	7	4	7	1	6	2	2	13.75	7	0	0	0	0	0	0	0	1	2	0	1	x1	0	0	0
79	10	2	10	1	7	20	14	4	6	0	4	17.5	7	0	0	0	0	0	0	0	1	3	0	1	x1	0	0	0
80	15	1	14	5	6	20	12	4	5	0	6	18.75	7	1	0	0	0	0	3	0	0	3	1	0	0	0	0	0
81	19	2	14	5	5	20	10	4	5	1	2	15	13	1	0	0	0	0	0	0	1	1	2	0	2	0	0	0
82	28	2	14	12	5	0	8	4	3	1	2	12.5	13	1	0	0	0	0	0	0	1	1	2	0	2	0	0	0
83	22	2	14	7	5	10	8	4	5	1	2	15	13	1	0	0	0	0	0	0	1	1	2	0	2	0	0	0
84	17	1	33	15	1	0	5	4	0	6	0	12.5	13	1	0	0	0	0	0	1	0	1	0	0	2	0	0	-6
85	22	4	8	5	8	20	11	1	5	0	6	15	14	-50	0	0	0	0	0	0	1	1	2	0	2	0	0	0
86	22	8	9	6	8	20	8	1	5	2	2	12.5	15	0	0	0	0	0	0	0	1	0	0	0	2	0	0	0
87	17	5	13	12	6	16	11	1	2	0	2	6.25	15	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0
88	0	1	33	10	1	0	20	4	3	0	4	13.75	12	0	0	0	0	0	0	1	0	2	0	0	0	0	0	0
89	0	1	24	6	3	20	20	4	5	0	4	16.25	12	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0
90	20	6	18	7	4	10	10	1	5	0	2	10	10	0	0	0	0	0	2	0	0	1	0	0	0	0	0	0
91	5	5	18	5	4	20	18	1	5	0	2	10	10	0	0	0	0	0	4	0	0	2	0	0	0	0	0	0
92	5	1	18	5	4	20	18	4	5	0	2	13.75	10	0	0	0	0	0	4	0	0	2	0	0	0	0	0	0
93	26	7	14	1	5	0	10	1	5	4	4	17.5	15	-5	0	0	0	0	0	0	2	4	0	3	x2	0	0	0
94	22	10	13	6	6	7	9	1	5	2	3	13.75	10	-25	0	3	0	0	0	0	0	2	2	0	0	0	0	0
95	28	6	22	3	3	0	7	1	6	1	2	12.5	7	0	0	0	0	0	0	0	2	4	0	0	0	0	0	0
96	19	0	7	2	10	20	10	6	6	4	6	27.5	8	0	0	0	0	0	2	0	0	2	0	0	0	0	0	0
97	24	6	7	3	10	18	8	1	6	0	3	12.5	14	0	0	0	0	0	0	2	0	1	0	0	0	0	0	0
98	19	1	8	5	8	20	12	4	5	1	2	15	10	0	0	0	0	0	0	0	1	2	0	0	2	0	0	0
99	11	2	8	5	8	20	13	4	5	1	2	15	10	0	0	0	0	0	0	0	1	2	0	0	2	0	0	0
100	22	2	18	6	4	18	9	4	5	0	2	13.75	14	0	0	0	0	0	0	0	0	1	0	0	3	0	0	0
101	17	6	18	8	4	20	13	1	5	0	2	10	10	-5	0	0	1	0	0	0	0	2	0	0	3	2	0	0
102	28	8	18	8	4	5	9	1	5	4	1	13.75	14	0	0	0	1	0	0	0	0	1	0	0	3	2	0	0
103	23	8	18	11	4	10	11	1	3	1	1	7.5	10	0	0	0	1	0	0	0	0	2	0	0	3	2	0	0
104	18	3	28	6	2	0	10	4	5	0	1	12.5	10	0	0	0	0	0	0	0	2	1	0	0	0	0	0	0
105	7	2	22	5	3	20	16	4	5	0	2	13.75	15	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0
106	35	1	22	5	3	20	16	4	5	0	2	13.75	15	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0
107	7	2	22	5	3	20	16	4	5	0	2	13.75	15	0	0	0	0	0	0	0	2	5	0	0	0	0	0	0
108	35	1	22	5	3	20	16	4	5	0	2	13.75	15	0	0	0	0	0	0	0	2	5	0	0	0	0	0	0
109	17	6	12	10	6	16	11	1	3	0	1	6.25	14	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
110	7	8	12	10	6	20	16	1	2	1	2	7.5	14	0	0	0	2	0	0	0	1	0	0	0	0	0	0	0
111	30	6	2	1	14	20	20	1	6	2	2	13.75	12	0	0	0	3	0	0	0	0	2	0	0	2	0	0	0
112	13	2	2	0	14	20	13	4	6	1	4	18.75	8	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0
113	26	10	2	0	14	10	8	1	6	4	4	18.75	8	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0
114	22	6	2	0	14	20	9	1	6	2	4	16.25	8	0	0	0	0	0	0	0	0	4	2	0	0	0	0	0
115	22	6	2	0	14	20	9	1	6	2	4	16.25	8	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0
116	20	13	6	3	11	20	10	0	6	4	0	12.5	8	0	0	0	0	0	0	0	3	3	0	0	0	0	0	0
117	8	15	6	3	11	20	16	0	6	0	2	10	8	0	0	0	0	0	0	0	2	3	0	0	0	0	0	0
118	6	2	2	1	14	20	17	4	6	4	2	20	15	-5	0	5	0	0	0	0	0	0	0	0	1	0	0	0
119	17	1	4	4	12	20	11	4	5	4	2	18.75	15	-5	0	5	0	0	0	0	2	0	0	0	1	0	0	0
120	22	1	4	4	12	20	9	4	5	4	2	18.75	15	-5	0	5	0	0	0	0	0	0	0	0	1	0	0	0
121	0	7	28	10	2	0	20	1	3	2	4	12.5	14	0	0	5	0	0	0	3	0	1	0	0	0	0	0	0
122	19	1	4	0	12	20	10	4	6	1	4	18.75	15	0	0	0	3	0	0	0	1	0	0	3	1	0	0	0
123	0	4	0	0	15	20	20	4	6	0	4	17.5	8	-50	0	0	0	0	3	0	0	4	0	0	0	0	0	0
124	11	5	0	0	15	20	14	1	6	0	4	13.75	13	0	0	0	0	0	2	0	0	2	0	0	0	0	0	0
125	2	1	1	4	15	20	20	4	5	0	6	18.75	10	0	0	0	3	0	0	0	2	2	0	0	0	0	0	0
126	31	5	1	0	15	0	5	1	6	1	2	12.5	4	0	0	0	3	0	0	0	2	5	0	0	0	0	0	0
127	23	5	1	0	15	5	14	1	6	2	2	13.75	8	0	0	0	3	0	0	0	2	4	0	0	0	0	0	0
128	28	3	1	0	15	0	7	4	6	0	2	15	12	0	0	0	3	0	0	0	2	1	0	0	0	0	0	0
129	19	5	1	0	15	16	12	1	6	0	5	15	12	0	0	0	3	0	0	0	2	1	0	0	0	0	0	0
130	19	2	1	0	15	20	10	4	6	1	2	16.25	10	-5	0	0	3	0	0	0	2	2	0	0	0	0	0	0
131	26	1	1	0	15	8	11	4	6	2	2	17.5	15	-5	0	0	3	0	0	0	2	0	0	0	0	0	0	0
132	24	2	1	0	15	18	8	4	6	4	2	20	13	0	0	0	3	0	0	0	1	1	0	0	0	0	0	0
133	16	2	1	0	15	20	12	4	6	1	2	16.25	13	0	0	0	3	0	0	0	1	1	0	0	0	0	0	0
134	14	2	1	0	15	20	12	4	6	1	2	16.25	13	0	0	0	0	0	0	1	1	1	0	0	0	0	0	0
135	12	1	2	6	14	20	13	4	5	1	6	20	13	-5	0	4	0	0	0	0	1	1	0	0	0	0	0	0
136	7	1	2	6	14	20	16	4	5	1	6	20	13	-5	0	4	0	0	0	0	1	1	0	0	0	0	0	0

APPENDIX 9.Ab RDA and RMR Ratings for field parameters

A	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA
63	0	0	0	1	0	5	0	0	0	0	0	1	0	-5	0	21	32	53	3	0	3	56	48	48
64	0	0	0	0	0	2	0	0	0	0	0	0	0	-2	0	31	32	63	1	0	1	64	57	57
65	0	0	0	0	0	0	0	0	0	0	0	-4	0	0	2	30	12	42	1	0	1	43	57	57
66	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	17	12	29	2	0	2	31	75	75
67	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	37	12	49	5	0	5	54	52	52
68	0	0	2	x1	0	0	0	0	0	-4	0	0	0	0	2	24	12	36	7	0	7	43	57	57
69	0	0	1	x1	0	0	0	0	0	-2	0	0	0	0	0	23	12	35	6	0	6	41	61	61
70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	32	43	75	8	0	8	83	26	26
71	0	0	0	2	0	0	-5	0	0	0	0	0	0	0	0	20	12	32	2	0	2	34	61	61
72	0	0	2	0	0	0	0	0	0	0	0	6	0	0	2	38	16	54	15	0	15	69	36	36
73	0	0	1	0	0	0	0	0	0	0	0	6	0	0	2	38	16	54	14	0	14	68	36	36
74	0	0	2	0	0	0	0	0	0	0	0	3	0	0	4	38	16	54	11	0	11	65	36	36
75	0	0	1	0	0	0	0	0	-3	0	0	0	0	-7	0	27	10	37	-5	0	-5	32	52	52
76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	25	10	35	5	0	5	40	66	67
77	0	0	0	0	0	0	0	0	0	0	0	4	0	0	2	29	10	39	11	0	11	50	58	59
78	0	0	0	0	0	2	0	0	0	0	0	0	0	0	4	32	11	43	10	0	10	53	39	39
79	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	12	11	23	7	0	7	30	66	66
80	0	0	0	0	2	3	0	0	-3	-3	0	0	0	-10	2	16	19	35	-2	0	-2	33	64	65
81	0	0	x1	1	0	0	0	0	0	0	0	0	0	0	0	21	19	40	7	0	7	47	63	64
82	0	0	x1	4	0	0	0	0	0	0	0	0	0	0	0	30	26	56	10	0	10	66	39	40
83	0	0	x1	1	0	0	0	0	0	0	0	0	0	0	2	24	21	45	9	0	9	54	51	52
84	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	18	48	66	-2	0	-2	64	32	33
85	0	0	0	3	0	0	0	0	0	0	0	-4	0	0	0	26	13	39	5	0	5	44	68	18
86	0	0	1	x1	0	0	0	0	-1	0	0	0	0	0	1	30	15	45	4	0	4	49	64	64
87	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	22	25	47	2	0	2	49	54	54
88	0	0	0	0	0	0	0	0	0	0	0	0	0	-4	0	1	43	44	-1	13	12	56	47	47
89	0	0	0	4	0	2	0	0	0	0	0	0	0	-7	0	1	30	31	1	0	1	32	71	71
90	0	0	0	1	0	0	0	0	-1	-7	0	0	0	-10	0	26	25	51	-14	0	-14	37	44	44
91	0	0	0	0	0	2	0	0	-3	-7	0	0	0	-10	1	10	23	33	-11	0	-11	22	62	62
92	0	0	0	0	0	2	0	0	-3	-7	0	0	0	-10	1	6	23	29	-11	0	-11	18	66	66
93	0	0	0	0	0	0	0	0	0	0	-2	6	0	0	0	33	15	48	13	0	13	61	48	43
94	0	0	0	0	0	0	0	0	0	0	0	7	4	-3	2	32	19	51	17	0	17	68	46	21
95	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	34	25	59	9	0	9	68	30	30
96	0	0	x1	2	0	0	0	0	0	0	0	0	0	-8	0	19	9	28	-2	0	-2	26	76	76
97	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	30	10	40	5	0	5	45	63	63
98	0	0	1	0	0	0	0	0	0	0	0	0	0	-4	0	20	13	33	2	0	2	35	65	65
99	0	0	0	2	0	0	0	0	-2	0	0	0	0	0	0	13	13	26	5	0	5	31	66	66
100	0	0	x1	0	3	0	0	0	-3	0	0	0	0	-3	0	24	24	48	1	0	1	49	59	59
101	0	0	x1	2	0	0	0	0	0	0	0	0	2	-3	0	23	26	49	9	0	9	58	57	52
102	0	0	0	1	0	0	0	0	0	0	0	0	0	-3	1	36	26	62	6	0	6	68	46	46
103	0	0	x1	2	0	0	0	0	0	0	0	2	0	-3	0	31	29	60	9	0	9	69	43	43
104	0	0	0	3	0	0	0	0	-4	-2	0	0	0	-4	0	21	34	55	-4	0	-4	51	35	35
105	0	0	0	1	0	0	0	0	-3	-2	0	0	0	-10	0	9	27	36	-12	0	-12	24	68	68
106	0	0	0	1	0	0	0	0	-3	-7	0	-7	0	-10	0	36	27	63	-24	0	-24	39	68	68
107	0	0	x2	4	x3	0	0	0	-3	-2	0	0	0	-10	0	9	27	36	-4	0	-4	32	68	68
108	0	0	x2	4	x3	0	0	0	-3	-7	0	-5	0	-10	0	36	27	63	-14	0	-14	49	68	68
109	0	0	0	0	0	0	0	0	0	-4	-2	0	0	0	2	23	22	45	-2	0	-2	43	53	53
110	0	0	0	0	0	0	0	0	-3	-2	0	0	0	-10	0	15	22	37	-12	0	-12	25	64	64
111	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	36	3	39	10	13	23	62	80	80
112	0	0	0	1	0	5	0	0	0	0	0	0	0	-4	0	15	2	17	6	0	6	23	74	74
113	0	0	0	1	0	5	0	0	0	0	0	0	0	-4	0	36	2	38	6	13	19	57	59	59
114	0	0	0	1	0	3	0	0	0	0	0	0	0	-4	2	28	2	30	8	0	8	38	67	67
115	0	0	0	1	0	0	0	0	0	0	0	0	0	-4	0	28	2	30	1	0	1	31	67	67
116	0	0	0	0	0	2	0	0	0	0	0	5	0	-5	1	33	9	42	9	0	9	51	62	62
117	0	0	0	0	0	2	0	0	-3	0	0	0	0	-5	2	23	9	32	1	0	1	33	65	65
118	0	0	0	0	0	2	0	0	0	0	0	7	0	0	2	8	3	11	17	0	17	28	86	81
119	0	0	0	0	0	0	0	0	0	0	0	7	0	0	2	18	8	26	17	0	17	43	77	72
120	0	0	0	0	0	2	0	0	0	0	0	7	0	0	2	23	8	31	17	0	17	48	75	70
121	0	0	0	0	0	0	0	0	0	0	0	0	0	-10	0	7	38	45	-1	9	8	53	49	49
122	0	0	0	0	0	0	0	0	0	0	-3	3	0	0	0	20	4	24	8	0	8	32	76	76
123	0	0	0	5	x4	2	0	0	-2	0	0	0	0	-5	0	4	0	4	7	0	7	11	81	31
124	0	0	0	0	0	5	0	0	0	0	0	0	6	-5	3	16	0	16	13	0	13	29	76	76
125	0	0	1	0	0	0	0	0	0	0	0	7	0	-6	0	3	5	8	9	0	9	17	84	84
126	0	0	0	0	0	0	0	0	0	0	0	0	0	-6	0	36	1	37	4	13	17	54	37	37
127	0	0	0	0	0	0	0	0	0	0	0	7	0	-6	0	28	1	29	10	0	10	39	56	56
128	0	0	0	0	0	0	0	0	0	0	0	3	0	-6	0	31	1	32	3	0	3	35	49	49
129	0	0	0	0	0	0	0	0	0	-3	0	0	0	-6	0	24	1	25	-3	0	-3	22	70	70
130	0	0	2	0	0	0	0	0	0	0	0	2	2	-6	0	21	1	22	7	0	7	29	71	66
131	0	0	0	0	0	0	0	0	0	0	0	-7	2	-6	0	27	1	28	-6	0	-6	22	67	62
132	0	0	1	0	x1	0	0	0	0	0	0	0	1	-10	0	26	1	27	-3	0	-3	24	74	74
133	0	0	1	0	x1	0	0	0	0	0	0	0	1	-10	0	18	1	19	-3	0	-3	16	76	76
134	0	0	1	0	x1	2	0	0	0	0	0	2	1	-10	0	16	1	17	-1	0	-1	16	76	76
135	0	0	0	1	0	0	0	0	0	0	0	6	0	-3	0	13	8	21	10	0	10	31	80	75
136	0	0	0	1	0	0	0	0	0	0	0	0	0	-3	0	8	8	16	4	0	4	20	83	78

APPENDIX 9.Ab RDA and RMR Ratings for field parameters

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC
137	19	10	2	2	15	20	10	1	6	2	2	13.75	15	-5	0	0	0	0	0	2	0	0	1	0	0	0	0	0
138	24	11	2	2	15	16	8	0	6	2	2	12.5	15	0	0	0	0	0	0	2	0	0	1	0	0	0	0	0
139	18	8	2	2	15	20	10	1	6	2	2	13.75	15	0	0	0	0	0	0	2	0	0	1	0	0	0	0	0
140	18	13	2	2	15	20	10	0	6	0	2	10	15	0	0	0	0	0	0	2	0	0	1	0	0	0	0	0
141	28	10	2	2	15	12	7	1	6	1	2	12.5	15	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
142	26	12	4	7	13	20	9	0	5	4	6	18.75	15	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0
143	26	12	4	7	13	20	9	0	5	4	6	18.75	15	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
144	26	12	4	7	13	20	9	0	5	4	6	18.75	15	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
145	5	3	1	2	15	20	18	4	6	0	6	20	9	0	0	0	0	0	0	0	2	2	0	0	0	0	0	0
146	22	4	1	2	15	18	9	4	6	6	6	27.5	9	0	0	0	0	0	0	0	2	2	0	0	0	0	0	0
147	12	2	11	4	7	20	13	4	6	0	6	20	10	-25	0	0	0	0	0	0	1	2	0	0	0	0	0	0
148	22	5	11	4	7	18	9	1	6	0	6	16.25	7	0	0	0	0	0	2	0	0	3	0	0	0	0	0	0
149	18	4	2	1	15	20	10	4	6	0	3	16.25	10	-50	0	0	0	0	4	0	0	2	0	0	0	0	0	0
150	15	4	11	1	7	10	12	4	6	1	6	21.25	9	-25	0	0	0	0	0	0	2	3	0	0	0	0	0	0
151	22	8	12	6	6	20	9	1	5	2	2	12.5	15	0	0	0	0	0	3	0	0	0	2	0	0	0	0	0
152	5	3	12	6	6	20	18	4	5	0	3	15	15	0	0	0	0	0	3	0	0	0	2	0	0	0	0	0
153	22	1	1	2	15	20	9	5	6	1	4	20	5	0	0	0	0	0	0	0	3	3	0	0	0	0	0	0
154	20	1	7	2	10	18	10	4	6	0	3	16.25	3	-25	0	0	0	0	0	0	2	5	0	0	0	0	0	0
155	10	1	7	2	10	20	14	4	6	0	3	16.25	3	-25	0	0	0	0	0	0	2	5	0	0	0	0	0	0
156	5	1	7	2	10	20	18	4	6	0	3	16.25	3	0	0	0	0	0	0	0	2	3	0	0	0	0	0	0
157	20	1	7	2	10	18	10	4	6	0	3	16.25	3	-5	0	0	0	0	0	0	1	5	0	0	0	0	0	0
158	35	1	20	10	3	0	20	5	3	6	6	25	15	0	0	0	0	0	0	0	1	0	0	2	1	0	0	0
159	26	7	5	0	11	14	7	1	6	2	6	18.75	15	0	0	0	0	0	0	0	1	0	0	2	1	0	0	0
160	23	4	5	0	11	18	9	4	6	2	6	22.5	15	0	0	0	0	0	0	0	1	0	0	2	1	0	0	0
161	24	5	3	5	13	16	8	1	5	2	6	17.5	10	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
162	17	1	4	5	12	20	11	4	5	0	3	15	12	0	0	0	0	0	0	0	3	1	0	0	0	0	0	0
163	17	1	4	5	12	20	11	4	5	0	3	15	12	0	0	0	0	0	0	0	3	1	0	0	0	0	0	0
164	17	1	4	5	12	20	11	4	5	0	3	15	12	0	0	0	0	0	0	0	3	2	0	0	0	0	0	0
165	29	2	6	0	11	0	12	4	6	4	3	21.25	12	0	0	0	3	0	0	0	0	0	0	0	2	0	0	0
166	12	4	6	0	11	20	13	1	6	0	3	12.5	12	0	0	0	3	0	0	0	0	0	0	0	1	0	0	0
167	24	6	6	0	11	18	8	1	6	0	3	12.5	12	0	0	0	3	0	0	0	0	0	0	0	1	0	0	0
168	28	9	6	0	11	18	7	1	6	0	3	12.5	12	0	0	0	3	0	0	0	0	0	0	0	1	0	0	0
169	26	5	7	5	9	10	7	1	5	0	6	15	15	0	0	0	0	0	0	0	1	0	0	0	1	0	0	0
170	32	5	12	5	6	0	5	1	5	6	4	20	15	0	0	0	0	0	0	0	1	0	0	0	1	0	0	0
171	5	1	1	1	15	20	18	4	6	4	2	20	7	0	0	0	2	0	0	0	2	3	0	0	2	0	0	0
172	24	6	12	0	6	20	8	1	6	0	2	11.25	4	0	0	0	0	0	0	0	3	5	0	0	2	0	0	0
173	31	5	18	7	4	0	6	1	5	4	2	15	11	-25	0	0	0	2	0	0	2	1	0	0	0	0	0	0
174	26	1	7	6	10	4	8	4	5	2	2	16.25	11	0	0	0	0	0	0	0	2	1	0	0	0	0	0	0
175	0	0	4	5	13	20	20	6	5	0	2	16.25	4	-25	0	0	0	0	4	0	0	6	0	0	0	0	0	0
176	31	0	2	5	14	20	11	6	5	0	6	21.25	15	-25	0	0	0	0	0	0	3	3	0	0	0	0	0	0
177	31	0	2	5	14	20	11	6	5	0	6	21.25	4	0	0	0	0	0	3	0	0	5	0	0	0	0	0	0
178	31	0	2	5	14	20	11	6	5	0	6	21.25	7	0	0	0	0	0	4	0	0	1	0	0	0	0	0	0
179	0	1	2	2	14	20	20	4	6	0	4	17.5	10	0	0	0	2	0	0	0	2	2	0	0	1	0	0	0
180	11	5	2	2	14	20	14	1	6	1	4	15	10	0	0	0	2	0	0	0	2	2	0	0	1	1	0	0
181	26	7	14	3	5	7	9	1	6	2	4	16.25	10	0	0	0	2	0	0	0	2	2	0	0	1	1	0	0
182	22	6	7	2	10	14	9	1	6	2	4	16.25	10	0	0	0	2	0	0	0	2	2	0	0	1	0	0	0
183	22	8	3	2	13	20	9	1	6	4	6	21.25	9	0	0	0	0	0	0	0	2	2	0	0	0	0	0	0
184	35	0	3	2	13	0	20	6	6	2	6	25	9	0	0	0	0	0	0	0	2	2	0	0	0	0	0	0
185	0	0	3	2	13	20	20	6	6	1	6	23.75	9	-25	0	0	0	0	0	0	2	2	0	0	0	0	0	0
186	26	2	10	6	7	7	8	4	5	2	6	21.25	15	0	0	0	0	0	0	2	0	0	0	0	1	0	0	0
187	17	6	7	3	10	20	10	1	6	1	2	12.5	15	-5	0	0	0	0	0	2	0	0	0	0	1	0	0	0
188	28	5	8	5	9	0	7	1	5	6	4	20	10	0	0	0	0	0	0	2	0	1	0	0	1	0	0	0
189	10	1	2	2	15	20	14	4	6	0	6	20	7	0	0	0	0	0	0	0	2	3	0	0	1	0	0	0
190	26	7	2	2	15	5	14	1	6	4	6	21.25	7	0	0	0	0	0	0	0	2	3	0	0	1	0	0	0
191	24	1	4	2	12	16	14	5	6	0	5	20	12	0	0	0	0	0	0	0	1	1	0	0	1	0	0	0
192	35	2	20	5	3	0	5	4	5	6	5	25	12	0	0	0	0	0	0	0	1	1	0	0	1	0	0	0
193	22	10	13	4	6	7	9	0	6	1	2	11.25	13	0	0	0	3	0	0	0	1	0	0	0	1	2	0	0
194	29	1	9	4	8	0	6	4	6	4	2	20	10	0	0	0	3	0	0	0	1	0	0	0	1	3	0	0
195	20	2	4	2	12	16	10	4	6	1	2	16.25	8	0	0	0	3	0	0	0	2	2	0	0	0	3	0	0
196	28	5	13	4	6	0	6	1	6	2	2	13.75	8	0	0	0	3	0	0	0	2	2	0	0	0	3	0	0
197	24	7	4	2	12	7	8	1	6	1	2	12.5	13	-5	0	0	3	0	0	0	1	0	0	0	1	1	0	0
198	13	2	4	2	12	14	13	4	6	1	2	16.25	9	-25	0	0	3	0	0	0	1	1	0	0	1	3	0	0
199	19	0	6	2	11	20	10	6	6	0	6	22.5	14	0	0	0	1	0	0	0	1	1	0	0	0	0	0	0
200	19	0	6	2	11	20	10	6	6	0	6	22.5	7	0	0	0	1	0	0	0	2	3	0	0	0	0	0	0
201	24	4	2	3	15	20	10	4	6	0	3	16.25	13	0	0	0	0	0	0	0	2	1	0	0	0	0	0	0
202	28	0	2	5	14	20	11	6	5	0	4	18.75	4	0	0	0	0	0	4	0	0	5	0	0	0	0	0	0
203	27	8	10	3	7	5	9	1	6	2	3	15	8	-5	0	5	0	0	0	0	2	3	0	0	1	0	0	0
204	10	1	4	1	12	20	14	5	6	0	6	21.25	14	-25	0	0	2	0	0	0	1	0	0	0	0	0	0	0
205	22	1	3	0	13	20	9	4	6	0	2	15	10	0	0	0	0	1	0	0	1	1	0	0	0	0	0	0
206	27	1	4	0	12	7	20	4	6	2	6	22.5	10	0	0	0	0	1	0	0	1	1	0	0	0	0	0	0
207	8	1	3	0	14	20	16	5	6	0	6	21.25	13	0	0	0	0	1	0	0	1	0	0	0	0	0	0	0
208	29	1	13	7	6	4	6	4	5	4	5	22.5	10	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0
209	28	1	6	2	11	0	7	4	6	2	2	17.5	14	0	4	0	0	0	0	0	2	0	0	0	0	0	0	0
210	12	1	6	2	11	20	13	4	6	1	2	16.25	10	0	4	0	0	0	0	0	2	2	0	0	0	0	0	0

APPENDIX 9.Ab RDA and RMR Ratings for field parameters

A	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA
137	0	0	x2	1	0	0	0	0	0	0	-3	0	1	-3	0	29	4	33	-1	0	-1	32	74	69
138	0	0	0	0	0	0	0	0	-2	0	-2	0	0	-3	0	35	4	39	-4	0	-4	35	67	67
139	0	0	0	0	0	0	0	0	-2	0	-2	0	0	-3	0	26	4	30	-4	0	-4	26	74	74
140	0	0	0	0	0	0	0	0	-2	-5	-2	0	0	-3	0	31	4	35	-9	0	-9	26	70	70
141	0	0	1	0	0	2	0	0	0	0	0	0	0	-3	0	38	4	42	1	9	10	52	62	62
142	0	0	0	0	0	5	0	0	0	0	-2	-5	0	0	0	38	11	49	1	5	6	55	76	76
143	0	0	0	0	0	5	0	0	0	0	-2	-5	0	0	0	38	11	49	2	5	7	56	76	76
144	0	0	0	0	0	5	0	0	0	0	-2	-5	0	0	0	38	11	49	0	5	5	54	76	76
145	0	0	0	0	0	2	0	0	-2	0	0	0	0	-7	0	8	3	11	-3	0	-3	8	82	82
146	0	0	0	0	0	2	0	0	-2	0	0	0	0	-7	0	26	3	29	-3	0	-3	26	79	79
147	0	0	0	0	0	0	0	0	0	-3	0	0	0	-5	0	14	15	29	-5	0	-5	24	70	45
148	0	0	0	0	0	-3	0	0	0	0	0	0	0	-10	0	27	15	42	-8	0	-8	34	57	57
149	0	0	0	0	2	3	0	0	0	0	0	0	6	-10	0	22	3	25	7	0	7	32	71	21
150	0	0	0	1	0	0	0	0	0	0	0	3	5	0	0	19	12	31	14	0	14	45	59	34
151	0	0	0	1	0	2	0	0	0	0	0	4	0	0	0	30	18	48	12	0	12	60	63	63
152	0	0	0	1	0	2	0	0	0	0	0	0	0	0	0	8	18	26	8	0	8	34	74	74
153	0	0	x3	4	x3	0	0	0	0	0	-5	0	0	-5	0	23	3	26	0	0	0	26	69	69
154	0	0	0	3	x2	0	0	0	-3	0	0	0	0	-5	0	21	9	30	2	0	2	32	57	32
155	0	0	0	3	x2	0	0	0	0	0	0	0	0	-5	1	11	9	20	6	0	6	26	63	38
156	0	0	0	3	x2	0	0	0	0	0	0	0	0	-5	0	6	9	15	3	0	3	18	67	67
157	0	0	0	0	0	0	0	0	-1	0	0	0	0	-5	0	21	9	30	0	0	0	30	57	52
158	0	0	0	0	0	2	0	0	0	0	0	-7	0	0	0	36	30	66	-1	0	-1	65	63	63
159	0	0	0	0	0	2	0	0	0	0	0	7	0	0	0	33	5	38	13	0	13	51	66	66
160	0	0	0	0	0	2	0	0	0	0	0	7	0	0	0	27	5	32	13	0	13	45	76	76
161	0	0	0	0	0	2	0	0	0	0	0	5	0	0	0	29	8	37	11	0	11	48	65	65
162	0	0	0	0	0	2	0	0	-3	0	0	-5	0	-5	0	18	9	27	-7	0	-7	20	70	70
163	0	0	2	0	0	2	0	0	0	0	0	-5	0	-5	0	18	9	27	-2	0	-2	25	70	70
164	0	0	x2	4	0	2	0	0	-3	0	0	-5	0	-5	0	18	9	27	-2	0	-2	25	70	70
165	0	0	0	0	0	0	0	0	0	0	0	0	0	-3	0	31	6	37	2	0	2	39	56	56
166	0	0	0	0	0	3	0	0	0	0	0	0	0	-3	0	16	6	22	4	0	4	26	69	69
167	0	0	0	0	0	0	0	0	0	0	0	0	0	-3	0	30	6	36	1	0	1	37	62	62
168	0	0	0	0	0	0	0	0	0	0	0	0	0	-3	0	37	6	43	1	9	10	53	61	61
169	0	0	x1	2	0	0	0	0	0	-7	0	7	0	-2	0	31	12	43	2	0	2	45	56	56
170	0	0	0	0	0	0	0	0	0	0	0	0	0	-2	0	37	17	54	0	0	0	54	46	46
171	0	0	0	0	0	0	0	0	0	0	0	7	0	-3	0	6	2	8	13	0	13	21	80	80
172	0	0	0	2	0	2	0	0	0	0	0	7	0	0	0	30	12	42	21	0	21	63	49	49
173	0	0	0	2	0	0	0	0	0	0	0	0	0	-3	0	36	25	61	4	0	4	65	36	11
174	0	0	3	0	0	0	-2	0	0	0	0	7	0	-5	0	27	13	40	6	0	6	46	49	49
175	0	0	0	0	0	5	0	0	-2	0	0	4	0	-10	3	0	9	9	10	0	10	19	73	48
176	0	0	0	x2	5	0	0	0	-2	-7	-2	-7	0	-10	0	31	7	38	-17	0	-17	21	81	56
177	0	0	0	x2	5	0	0	0	-2	-6	-2	-7	0	-10	0	31	7	38	-14	0	-14	24	70	70
178	0	0	0	x1	2	0	0	0	-2	-6	-2	-7	0	-10	0	31	7	38	-20	0	-20	18	73	73
179	0	0	0	1	0	2	0	0	-2	0	0	0	0	-3	0	1	4	5	5	0	5	10	82	82
180	0	0	0	1	0	2	0	0	-2	0	0	0	0	-3	0	16	4	20	6	0	6	26	73	73
181	0	0	0	1	0	2	0	0	-2	0	0	0	0	-3	0	33	17	50	6	0	6	56	47	47
182	0	0	0	1	0	2	0	0	-2	0	0	0	2	-3	0	28	9	37	7	0	7	44	59	59
183	0	0	1	0	0	2	0	0	0	0	0	-4	0	-7	0	30	5	35	-4	0	-4	31	72	72
184	0	0	1	0	0	2	0	0	0	0	0	0	0	-7	0	35	5	40	0	0	0	40	67	67
185	0	0	1	0	0	2	0	0	0	0	0	0	6	-7	0	0	5	5	6	0	6	11	86	61
186	0	0	0	0	0	0	0	0	0	0	0	0	0	-2	0	28	16	44	1	0	1	45	58	58
187	0	0	0	2	0	3	0	0	0	0	-3	-2	0	-2	0	23	10	33	1	0	1	34	68	63
188	0	0	0	0	0	3	0	0	0	0	-3	0	0	-2	0	33	13	46	2	0	2	48	46	46
189	0	0	0	0	0	0	0	0	0	0	0	0	0	-3	0	11	4	15	3	0	3	18	76	76
190	0	0	x1	1	0	0	0	0	0	0	0	4	0	-3	0	33	4	37	8	0	8	45	62	62
191	0	0	1	0	0	0	0	0	0	-7	0	0	0	-3	0	25	6	31	-6	0	-6	25	74	74
192	0	0	1	0	0	0	0	0	0	0	0	0	0	-3	0	37	25	62	1	0	1	63	45	45
193	0	0	0	0	0	0	0	0	0	0	0	-2	0	-3	0	32	17	49	2	0	2	51	46	46
194	0	0	1	0	0	0	0	0	0	0	0	0	0	-3	0	30	13	43	6	0	6	49	44	44
195	0	0	3	x2	0	0	0	0	0	-3	0	7	0	-3	0	22	6	28	14	0	14	42	62	62
196	0	0	2	0	0	0	0	0	0	0	0	0	0	-3	0	33	17	50	9	0	9	59	34	34
197	0	0	0	0	0	0	0	0	0	0	0	2	0	-3	0	31	6	37	5	0	5	42	53	48
198	0	0	0	0	0	0	0	0	0	0	0	5	1	-5	0	15	6	21	10	0	10	31	64	39
199	0	0	0	0	0	3	0	0	-3	-7	0	0	0	-4	0	19	8	27	-8	0	-8	19	78	78
200	0	0	0	0	0	3	0	0	-3	-7	0	0	0	-4	0	19	8	27	-5	0	-5	22	71	71
201	0	0	2	0	0	0	0	0	0	-7	0	-5	0	-10	0	28	5	33	-17	0	-17	16	74	74
202	0	0	0	0	0	3	0	0	-2	-7	-2	-7	0	-10	0	28	7	35	-16	0	-16	19	68	68
203	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	35	13	48	11	0	11	59	44	39
204	0	0	3	x2	x1	3	0	0	-1	0	0	2	0	-10	0	11	5	16	0	0	0	16	81	56
205	0	0	x1	1	0	2	0	0	0	0	0	2	0	0	0	23	3	26	8	0	8	34	67	67
206	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	28	4	32	3	0	3	35	72	72
207	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9	3	12	2	0	2	14	84	84
208	0	0	0	1	0	0	0	0	-2	0	-5	0	0	-5	0	30	20	50	-9	0	-9	41	49	49
209	0	0	0	0	0	0	0	0	0	-5	0	0	0	-10	0	29	8	37	-9	0	-9	28	50	50
210	0	0	3	x2	x1	0	0	0	0	-3	0	0	0	-10	0	13	8	21	-2	0	-2	19	70	70

APPENDIX 9.Ab RDA and RMR Ratings for field parameters

Site number	Rock mass type	Subsidiary rock mass	Intensely fractured zones	Grain ravelling	Stone ravelling	Block ravelling	Flaking	Wash erosion	Solution	Karstification	Flexural toppling	Grainfall	Stone fall	Block fall	Scaling	Slabfall	Toppling	Rockfall	Debris flow	Rockslide
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
1	L			2				2												
2	L							1					2							
3	B							1							2			1		
4	L			1				1						1						
5	L			1				1						1	2	1				
6	L			1				1						1						
7	L			1	1			1					2	1						
8	L			1				1						1						
9	L			1																
10	L			1																
11	L							2	2											
12	L								1											
13	L/B																	1		
14	Lc				1			1					2	1	2	1		2		
15	Lc/B							1					2	1	2	1				
16	Mw	C		2								2		1	1			2		
17	Mw	L											1							
18	B	Bi												2	2	2		2		
19	Mw	L											1							
20	B	Bi												2	2	2				
21	Lf				1		2						1					2		
22	L												1	1	1					
23	B				2									2				1		
24	L	Bi							1											
25	Mw/L			2	2							2	2	1	2	1		1		
26	Mw							1				2	1							
27	Ms												1							
28	B		Y		2		1									1				
29	L/Bi												1							
30	L/Bi												1							
31	L/Bi												1							
32	Mw			2				1					1		2					
33	Mw			2				2					1		2					
34	Lc	B	Y		2			1						2						
35	L/Lc	B	Y		2			1						2						
36	Ms												1		2					
37	Lc	B			2	2								2				2		
38	Lf						2	2					1							
39	L			1				2					1							
40	L				1			1	1				2							
41	L								2											
42	L							1	1				2							
43	B		Y		2			1	1											
44	L/B				1			1					2	2		1		2		
45	B		Y	2	2			2	2							1	1	1		
46	Lf/Lc						2	2					2	2				1		
47	B		Y	2	2								2	2				2		
48	L												1							
49	L/B				2								1	2						
50	B				1	2							1	2				1		
51	L	B					2		1				1	2	1		2			
52	Ms								1					1			2			
53	L	B	Y		1						1		2				2			
54	L				1		1											2		
55	Lc		Y		2	2												2		
56	Ms								1					1						
57	B							1					2	2	2			2		
58	B		Y				1							2	2					
59	L			2	2	1		2							2			2		
60	Ms/L			1				2				1		1	2					
61	L/B		Y		2	1	1						2	1	2			2		1
62	L/B				2		1						2	2						
63	L	Lc	Y		2				1					1		1		1		
64	B		Y		2								1	2						

APPENDIX 9.Ac Field occurrence of deterioration modes and rock mass types

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
65	B				2															
66	L/B													2				1		
67	B				2													2		
68	L			2				1					1							
69	B			2				1					1							
70	Lf		Y	2			2	1												
71	B			1				1					1		2					
72	B	L	Y	1	2		1											2		
73	B	L	Y	1	2		1											2		
74	B	L	Y	1	2		1		2	1								2		
75	L							2	2					2		1				
76	L							1	1					2						
77	L						2	2	1					2						
78	B												2					2		
79	B/Ms								2				1							
80	Ms	L		1								1	1							
81	L												1							
82	B				1			2												
83	L													2				1		
84	Mw							1				2								
85	L												2	1				1		
86	B			2	2			2					2							
87	L			2				2				2								
88	Mw			2				2				2	1							
89	Mw			1				2	1			1								
90	Ms							2					1							
91	Ms							2							2					
92	Ms							2							1					
93	Bi	C	Y	2	2	2							1	1				1		
94	L/Lc	Bp	Y		2		2								2					
95	B	R		2	2	1	1	1												
96	B	R							1				1	1						
97	B												1					2		
98	Bi		Y		2			2					1					1		
99	B				2								1							
100	L			2				1												
101	L/Bi		Y		1			2					1	2			1			
102	Bi/L	R	Y		2			2	1				2							
103	L/Bi	R	Y		2			2	1				2							
104	L	R			1								2		2	1				
105	Ms		Y	1				2				1			2			2		
106	L		Y	2				2				2			1					
107	Ms		Y					2					1							
108	L		Y					2					1							
109	L											2		1		2				
110	B			1								1	1		2					
111	B		Y	2	2			2							1			1		
112	B		Y											2				1		
113	B/Bi				2	1												2		
114	B				1									1						
115	B												1	2						
116	L/Bi	V		2	1									2				2		
117	Bi/Ms													2		1		2		
118	Bi	Pw		2			2	1					2	1						
119	Bi	Pw		2			2	1					2	1						
120	Bi	Pw		2			2	1					2	1						
121	L	P						2						1	2		1	2		
122	Bi		Y	2	1									1						
123	Ms														1					
124	Ms														1	2	1			
125	Ms/Bi							1					2							
126	L	V		2	2															
127	Bi		Y		2			2					2							
128	L	V			2			1					1							
129	B												1							
130	L	V											2				1	1		
131	B																	2		
132	B	Ms			1								2	1				1		
133	B	Ms			1								2							
134	B	Ms			1								2							
135	B											1		1				2		
136	Ms	C										2			2					
137	Bi																1			
138	Bi				2												1			

APPENDIX 9.Ac Field occurrence of deterioration modes and rock mass types

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
139	Bi													1				1		
140	Bi													1				1		
141	L	Bi	Y		2								2	2			1	1		
142	Bi				2													2	2	
143	Bi				2													2	2	
144	Bi				2													2	2	
145	Ms														2					
146	Ms													2	2					
147	L	P											2							
148	B												2							
149	L													2		2		2		
150	L				1	1								2		2		1		
151	B		Y		2	2								1						
152	B													2						
153	B	Ms											1			2		1		
154	Ms	Bi					2													
155	Ms	Bi					1													
156	Ms						1													
157	B/Bi						2						2							
158	Ms	B	Y	2			2													
159	B		Y		2								2					2		
160	L		Y											1		2	1			
161	B				2			2										2		
162	Ms												2	1						
163	Ms												2	1						
164	Ms		Y										2	1						
165	B		Y	2	1															
166	B				2															
167	B				2								1	1						
168	Bi			1	1													2		
169	L			2	2		2													
170	B		Y	2																
171	L/Lf	B	Y				1						1							
172	B	Lf	Y	2	2	1		1					1	1				1		
173	Lf	Bi	Y				2								2	1		2		
174	Lf	V	Y		1		2				2									
175	Ms	L					1											2		1
176	L	Lf					2													
177	L	Lf					2													
178	L	Lf					2													
179	Ms																			
180	L	V											1	1				1		
181	B				2			2					2							
182	L	V			2			2					2							
183	Bi/L													1		1				
184	B			2	1		2	1												
185	Ms															2	1			
186	L	V			2		2							1						
187	B												2							
188	B		Y		2															
189	Ms/Bi				2															
190	Bi		Y	2	2			1												
191	Bi	Ms			2									1		1				
192	Lf		Y	1			2	1												
193	B						2						2					2		
194	B/Bi							1					2					2		
195	L/B		Y		2								2	1				1		
196	Lf			2	1		2	2					1	1						
197	Bi		Y		2		1	1						2				2		
198	L/B		Y		2		2						2							
199	Lc						1							2						
200	Lc						1							2						
201	Lf	L					2									1				
202	L	Lf					1													
203	L	Lf		1	2	2		1						1				1		
204	L	Bi													1	1				
205	L/B							1						1						
206	B												2							
207	Ms/Bi													1						
208	Bi	Ms					2									1		1		
209	Lf	V	Y	2	1		2	1					1							
210	L/B	V			2		1	1					2	1						

APPENDIX 9.Ac Field occurrence of deterioration modes and rock mass types

APPENDIX 9.B

DATA USED IN THE EVALUATION OF INTER-RELATIONSHIPS BETWEEN RDA CLASS AND DETERIORATION MODE

Data given in the table below are the *percentage* of slope units for each RDA_A class in which the major deterioration modes listed occurred. The data was used to generate the charts shown in Figure 9.6 (a to h). The *absolute* frequency values pertaining to the data were given in Table 9.2.

Deterioration mode	**RDA_A Class**					
	1	**2**	**3**	**4**	**5**	**Total**
Stonefall	20	17	26	27	0	46
Stone ravelling	8	11	42	58	0	58
Blockfall	4	12	26	12	0	34
Wash	4	15	17	27	50	34
Rockfall	4	6	29	35	0	37
Grain ravelling	0	10	21	35	50	34
Flaking	8	9	13	23	100	27
Scaling	16	10	12	19	0	26
Slabfall	4	4	7	0	0	9
Solution	4	4	0	8	0	6
Grainfall	4	2	4	12	0	9
Toppling	0	1	3	0	0	3
Block ravelling	0	0	7	4	0	6
Debris flow	0	0	4	0	0	3
Flexural toppling	0	0	1	0	0	1
Rockslide	0	0	0	0	0	0
Karst	0	0	0	0	0	0
Total number of slope units	*25*	*81*	*76*	*26*	*2*	*210*

APPENDIX 9.C

RATINGS AND PARAMETERS FOR THE ROCK MASS RATING SYSTEM (*after Bieniawski 1979*)

(a) Classification Parameters and their Ratings

	Parameter		*Ranges of values*				
1	Strength of intact rock material	Point load strength index (MPa)	> 10	4–10	2–4	1–2	For this low range, uniaxial compressive test is preferred
		Uniaxial compressive strength (MPa)	> 250	100–250	50–100	25–50	5–25 1–5 < 1
		Rating	15	12	7	4	2 1 0
2	Drill core quality RQD (%)		90–100	75–90	50–75	25–50	< 25
		Rating	20	17	13	8	3
3	Spacing of discontinuities		> 2 m	0.6–2 m	200–600 mm	60–200 mm	< 60 mm
		Rating	20	15	10	8	5
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1–5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous
		Rating	30	25	20	10	0
5	Groundwater	Inflow per 10 m tunnel length (L min^{-1})	None	< 10	10–25	25–125	> 125
			or	or	or	or	or
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	< 0.1	0.1–0.2	0.2–0.5	> 0.5
			or	or	or	or	or
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
		Rating	15	10	7	4	0

A: The Rock Mass Rating system (Geomechanics Classification) after Bieniawski (*copied from Bieniawski 1993*)
Classification of parameters and their ratings

(b) Rating Adjustment for Discontinuity Orientations

Strike and dip orientations of discontinuities		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels and mines	0	−2	−5	−10	−12
	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	−60

(c) Rock Mass Classes Determined from Total Ratings

Rating	100–81	80–61	60–41	40–21	< 20
Class	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

(d) Meaning of Rock Mass Classes

Class	I	II	III	IV	V
Average stand-up time	20 y for 15 m span	1 y for 10 m span	1 week for 5 m span	10 h for 2.5 m span	30 min for 1 m span
Cohesion of the rock mass (kPa)	> 400	300–400	200–300	100–200	< 100
Friction angle of the rock mass (deg)	> 45	35–45	25–35	15–25	< 15

B: The Rock Mass Rating system (Geomechanics Classification) after Bieniawski (*copied from Bieniawski 1993*)
Rating adjustment for discontinuity orientations

Parameter[a]	*Ratings*				
Discontinuity length (persistence/continuity)	< 1 m	1–3 m	3–10 m	10–20 m	> 20 m
	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1–1.0 mm	1–5 mm	> 5 mm
	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
	6	5	3	1	0
		Hard filling		*Soft filling*	
Infilling (gouge)	None	< 5 mm	> 5 mm	< 5 mm	> 5 mm
	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
	6	5	3	1	0

[a] Some conditions are mutually exclusive. For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge. In such cases, use Table 2 directly.

C: The Rock Mass Rating system (Geomechanics Classification) after Bieniawski (*copied from Bieniawski 1993*)
Guidelines for classification of discontinuity condition

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